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March, June, September and December

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1956-1957

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GEORGE C. HOUSER, 220 Clyde Street, Brookline 67, Massachusetts.

THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of over 1000 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

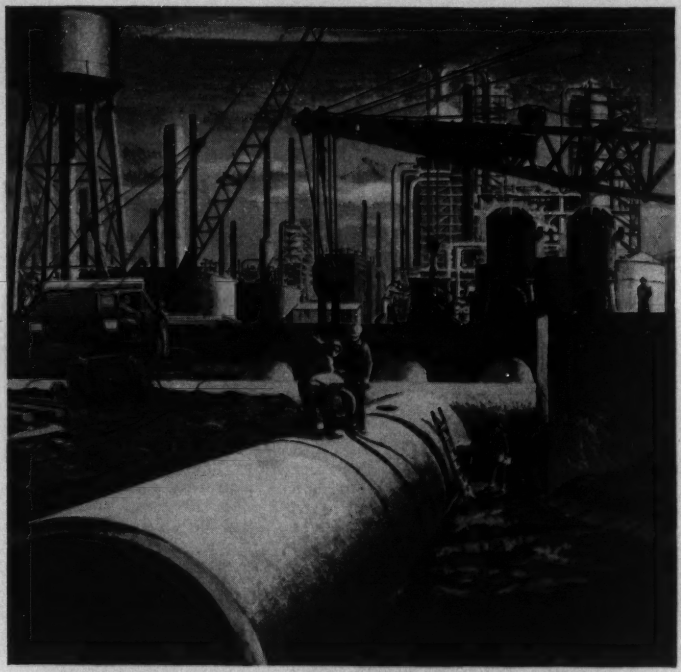
An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

Initiation Fees		*Annual Dues*	
Members	$ 5.00	Members	$ 5.00
Juniors	1.00	Juniors	2.00
Associate	10.00	Associate	25.00
Corporate Members	10.00	Corporate Members	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September or October on such date as the Executive Committee may designate.



Where OIL and WATER do mix!

It takes a staggering amount of water to run an oil refinery such as the one recently built for the Tidewater Oil Co. at Delaware City, Delaware. Specifications required twin 78" LOCK JOINT REINFORCED CONCRETE CYLINDER PIPELINES to supply water to this plant. The combined capacities of these two lines would satisfy the normal requirements of a city of over two million population.

Only the most rugged and dependable type of pipe could be trusted to handle such volumes of water within the limited confines of a multi-million dollar plant. Yet Tidewater authorities are confident of the durability and safety of the refinery's Lock Joint supply lines because these characteristics are built into every Lock Joint pressure pipe through conservative design, careful choice of materials and expert workmanship.

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By any comparison—accuracy, durability, repair ease or convenience—Rockwell meters measure up to the highest standards. Ask our representative to demonstrate, or write today for latest bulletin.

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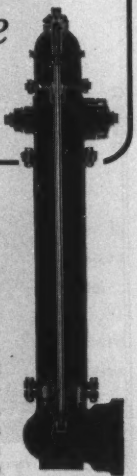
A Size and Type For Every Kind of Service

The name MATHEWS on hydrants means the finest in fire protection



Also available with stem coupling and breakable flange

THE MATHEWS MODERNIZED HYDRANT

The hydrant with the *replaceable* barrel. A Mathews broken in a traffic accident can be repaired in less than half an hour by inserting a new barrel without *excavating*. In addition this high-quality hydrant is very simply constructed. It is the *dry-head* type which functions under all conditions, because water, ice and sediment cannot reach key parts to clog operation.

THE MATHEWS FLANGE BARREL HYDRANT

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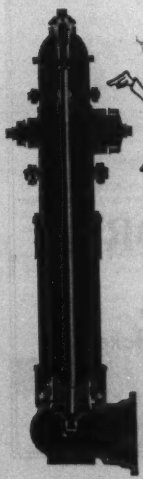
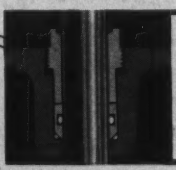


All available with mechanical joint and flange-type connections



All Mathews Hydrants are now available with O-Ring Seals when specified.

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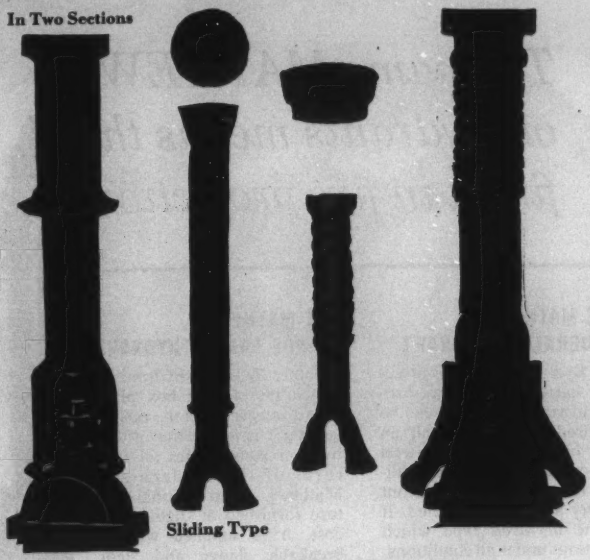

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Manhole Frames and Covers

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a few plain words about...

WATER METERS!

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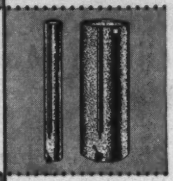
The Ford Yoke dates back to 1911. Its outstanding success resulted in a wide variety of meter holding devices called yokes. But the Ford yoke as now made offers advantages not obtainable in any other type of meter setting. Millions are in use.

LIFTER WORM LOCK

The original Ford Worm Lock was introduced almost forty years ago. Because of its simplicity, safety and powerful screw-jack action it is now widely used. The Lifter feature, a recent improvement, provides automatic attachment of the key on unlocking.

CALIBRATED TESTING TANKS

Before Ford Calibrated Testing Tanks were put on the market about 35 years ago, such meters as were tested were checked against gravimetric scales, which were complicated and inconvenient. Thousands of Ford Tanks are in service, in sizes from one to 1000 gallons.



METER VALVE

The Meter Valve is a close-coupled combination of an inverted key valve and a meter connection. It saves material, pipe joints and space in the meter setting. Ford Meter Valves bear the trademark name of Ringstyle Valves.

THE KORNERHORN

In a vertical basement pipe, against the wall or in a corner, the Kornerhorn makes a most satisfactory meter setting. Installation is fast and economical as pipe threading is eliminated. Send for detailed information.

Meters can *earn* real money. In most water utilities they measure and determine *all* the income.

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For 60 years our company has served thousands of water utilities in providing better water meter settings. Submit *your* problems to Ford.

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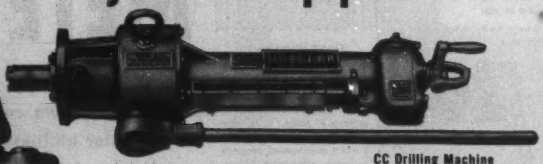
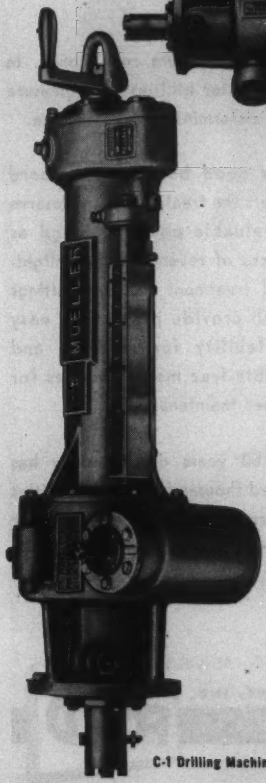


CC Drilling Machine



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- Hand or power operation
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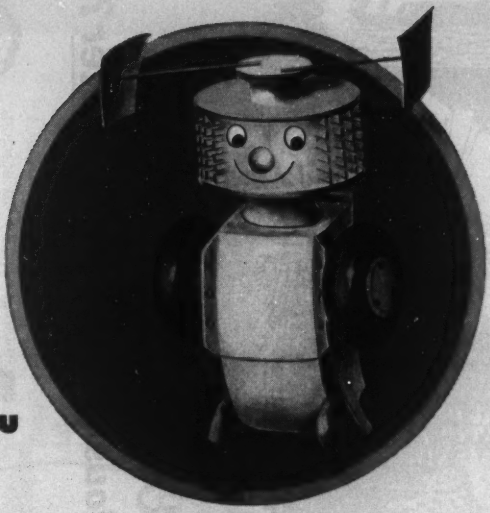

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If not, you should! He can do things for you . . . like making your job easier and saving you money. He's a versatile and somewhat ingenious many-armed creature who specializes in cement-mortar lining the inside of pipes. And he's done this to over 4 million feet of pipe, too—eliminating corrosion and tuberculation . . . increasing flow capacity . . . reducing leakage . . . slashing pumping costs . . . and adding years and years of pipe life. If you'd like to learn more about Dr. Centriline and how he can work for you, just write or call for one of our descriptive brochures.

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• CEMENT-MORTAR LINING OF PIPES IN PLACE •

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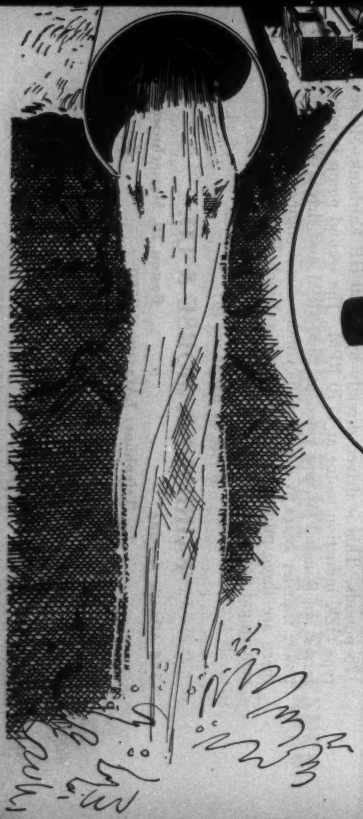
Forest conservation costs millions — and it's worth it. But the millions of *wasted gallons* of increasingly scarce water *could* cost the future of entire communities. Yet one basic step—replacing flat rates with Badger Meters



Outstanding disc, turbine and compound meters—⅝" through 12" —precision-engineered for lasting service.

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Complex fire fighting system guards world's largest Auto Assembly Plant. Ford Motor Company at Mahwah, N. J. Protected by extensive system including two Fairbanks-Morse Centrifugal Fire Pumps.

One of the newest in the Ford chain of 16 assembly plants, the giant Mahwah installation is capable of turning out more than 1,000 cars and trucks in a single two-shift working day.

The system was designed by Ford engineers in collaboration with consulting engineers R. F. Giffels and V. E. Vallet.

The pumps chosen for this important service are two Figure 5814F Fairbanks-Morse horizontal centrifugals. One is a 6-inch pump with rated capacity of 1500 gallons per minute at a discharge pressure of 125 pounds. This is driven directly by a 150 HP Fairbanks-Morse type QZK induction motor at 1770 RPM. The second pump is even larger, an 8-inch F-M unit rated at 2500 GPM and driven through power transfer gears by a pair of 6-cylinder diesel engines. Actually there are three pumps, the third being a 1-inch Figure 5592- F-M centrifugal driven at 3475 RPM by a 15 HP motor.

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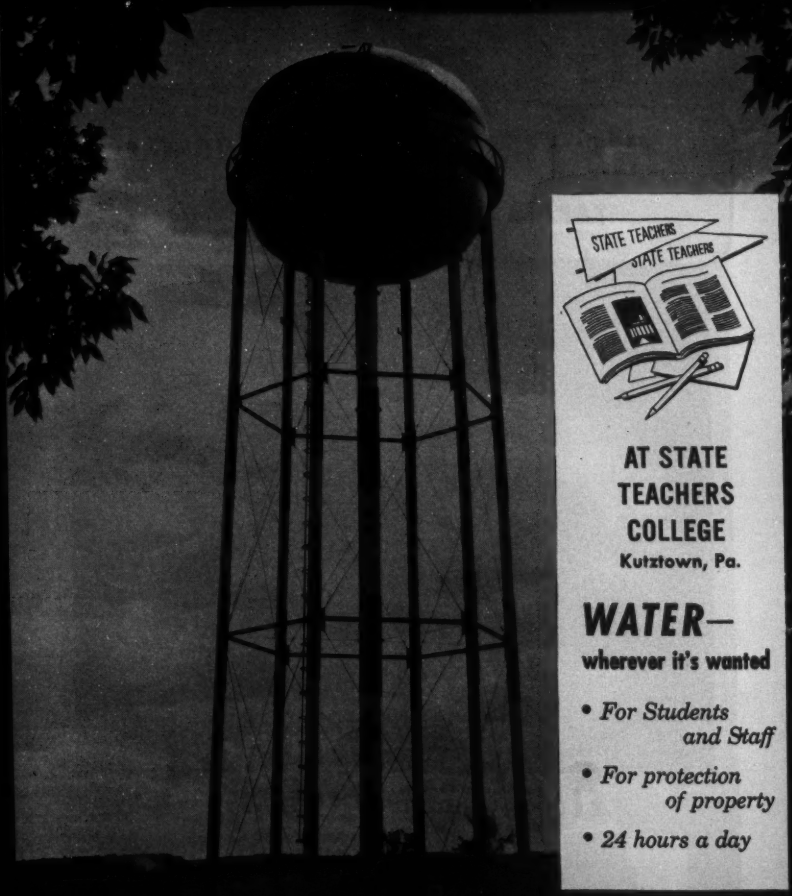

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wherever it's wanted

- *For Students and Staff*
- *For protection of property*
- *24 hours a day*

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Pittsburgh-Des Moines water storage with a "college degree" for dependability and economy serves this Eastern Pennsylvania school assignment. Built for the General State Authority, the double-ellipsoidal PDM elevated steel tank has a capacity of 500,000 gallons, a diameter of 44 ft and head-range of 30 ft 9 in. It is supported on a tubular column tower, 114 ft 6 in. to bottom of capacity.

Let us quote on your water storage requirements. Write for our latest Catalog.

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ELEVATED STEEL TANKS

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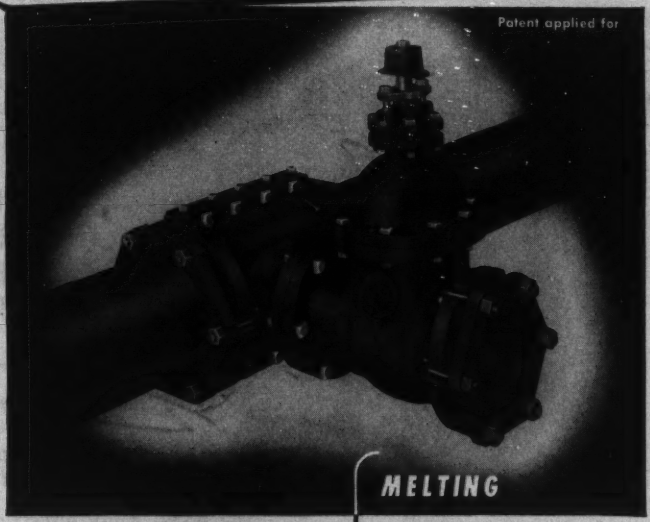
THE FIRST MECHANICAL JOINT TAPPING SLEEVE AND VALVE



ELIMINATES { MELTING / POURING / CAULKING

A new *all* Mechanical Joint Tapping Sleeve and Valve, installed quickly and easily even by untrained workmen using a ratchet wrench. There are no joints to pour and caulk.
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The "Stuffing Box" type joints require no cutting or scarfing of rubber gaskets on job. Assemble Sleeve on main—attach the Valve, the installation is complete and the joints are permanently bottle-tight.

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GATE VALVES
CHECK VALVES
FIRE HYDRANTS
INSERTING VALVES
TAPPING SLEEVES AND VALVES
WATER SERVICE BRASS GOODS

THE A. P. SMITH MFG. CO.
ESTABLISHED 1896
EAST ORANGE, NEW JERSEY

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Like hundreds of Pollard customers, you'll find it mighty pleasant and convenient to order pipe line equipment from Pollard.

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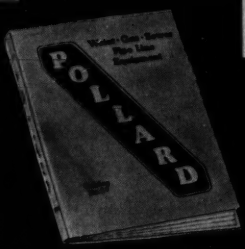

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Our contract included installation of pumps, chemical equipment, intermediate piping and electrical control equipment.

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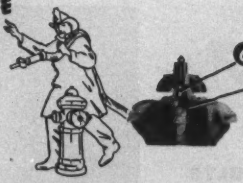
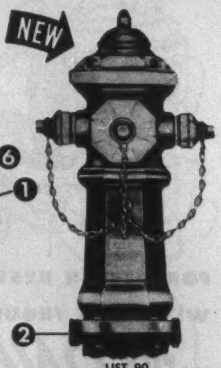
EASY TO OPERATE AND MAINTAIN





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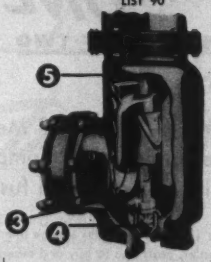

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Ludlow hydrants incorporate all the advantages and meet all the requirements for safety, durability and economy.

HERE'S WHY:

1. **POP-OFF SLEEVE COUPLING:** releases hydrant head from stem in event of traffic damage.
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5. **90,000 TENSILE STRENGTH** rolled Everdur threaded stem section completely eliminates stem failure.
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THE RESULT: LOW INITIAL COST—YEAR AFTER YEAR—OF TROUBLE-FREE SERVICE.

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"NO POSSIBLE DIFFERENCE IN FIRST COST CAN OVERBALANCE THE PERPETUAL ECONOMY OF QUALITY."

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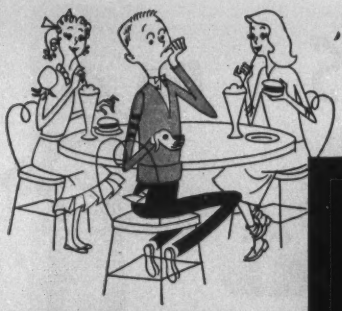
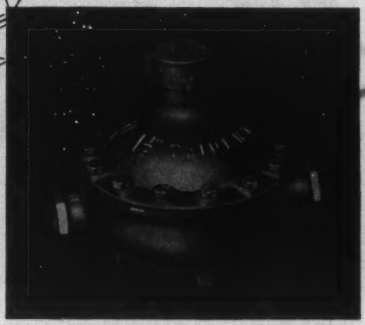
Since 1861 THE LUDLOW VALVE MANUFACTURING CO. Troy, N. Y.





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WITH LESS TROUBLE
...PICK *ONE*
INSTEAD OF TWO

1½" or 2" Style 3 Meter matches performance of complicated compound . . . with less cost and fuss

For 1½" and 2" water service lines, the Trident Style 3 meter is simpler, costs less to buy and maintain, is every bit as accurate, and produces just as much revenue over a wide range of flows as any compound, including our own. Trident was *first* to give you an easy-to-set pressure adjustment. And since modern Style 3 parts fit older meters, there's never any obsolescence.

So why put up with the fuss and expense of two measuring units when one Style 3 will do the job? You'll find conclusive evidence in your own records . . . or ask your Neptune man.

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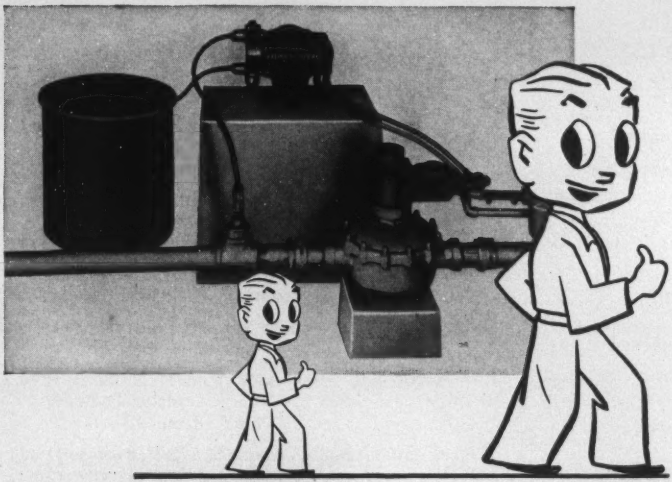

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For treating gravity fed water supplies, here's a natural. The only power required to drive Proportioneers Automatic and Proportional Chem-O-Feeder is the flow of water through the pipe line itself. The Chem-O-Feeder delivers water treating chemicals in strict proportion to the main flow: low flow . . . low feed — high flow . . . high feed.

Simple and dependable, the Chem-O-Feeder is paced by a standard main line, by-pass, or fire line meter — and is easily adapted to most any meter already in service. The meter does not drive the pump but serves simply as a control unit — thus the accuracy of the meter itself is in no way impaired. Bulletin SAN-6 gives complete details . . . send for your copy today. Proportioneers, Inc., 366 Harris Avenue, Providence 1, Rhode Island.

PROPORTIONEERS, INC.
DIVISION OF
B-I-F INDUSTRIES
PROVIDENCE, RHODE ISLAND
METERS FEEDERS CONTROLS

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Tentative Standard Specifications for Dimensions for Steel Water Pipe Fittings ..... Appended

EDWARD L. TRACY

President of the New England Water Works Association
1956 - 1957

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NEW PUMPING FACILITIES AT BROOKLINE, MASSACHUSETTS, FOR THE METROPOLITAN DISTRICT COMMISSION

BY JOHN C. ADAMS, JR.*

[*Read September 19, 1955.*]

Brookline has been a member of the Metropolitan Water District since 1925, although until just recently the town continued to utilize its own source of supply and pumping facilities on the Charles River. In recent years, due to the inability of the Brookline sources to yield a sufficient supply, it has been necessary for the town to rely on connections with the Metropolitan District to supplement its own supply. In 1948, the town decided to abandon its local supply and formally notified the Metropolitan District Commission of its desire to enter into an agreement to obtain all of its water from Metropolitan sources.

For many years, the towns of Dedham, Westwood, Norwood and Canton—none of which were members of the Metropolitan Water District, but all of which are eligible for membership—have been considering the possibilities of obtaining their supply from the Metropolitan. In recent years, many inquiries have been made by officials and other residents of these towns regarding the problems and costs involved in their joining the District.

As it was recognized that these two problems—(1) the immediate supply to Brookline and (2) the future supply to Dedham, Westwood, Norwood and Canton—were closely related and interdependent, the Metropolitan District Commission retained Coffin & Richardson in 1951 to develop a comprehensive plan for the future extension of the Southern High and Southern Extra High Service Distribution Systems.

The resulting plan was submitted to the Metropolitan District Commission in early 1952 and embodied some $14,000,000 of improve-

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ments and additions, which were divided into three construction phases. The Metropolitan's recently completed Newton Street Pumping Station—to give it its formal name—was a part of the first phase of this over-all plan.

The Southern High Service System originates at the present easterly extremity of the City Tunnel, which is supplied under the head of Norumbega Reservoir. From here the system extends southeasterly through Brookline, the West Roxbury and Dorchester sections of Boston, and Milton to the Blue Hills Reservoir in Quincy. The Southern Extra High Service System, as it existed in 1952, consisted of a pumping station in Hyde Park which pumped from the Southern High Service System to Bellevue Reservoir and supplied a small portion of Boston and Milton.

Studies showed that a portion of Brookline would require the higher head of the Southern Extra High Service System and that Dedham, Westwood, Norwood and Canton could also best be supplied with the higher head.

The resulting plan consists basically of the following new construction:

1. A 30-inch suction main from the Southern High Service System to a pumping station on Newton Street in Brookline.
2. A pumping station in Brookline which would work in conjunction with the present Hyde Park Pumping Station to supply the Southern Extra High Service System.
3. A 36-inch force main to a new, higher, and larger Bellevue Reservoir.
4. A greatly expanded distribution system south through Dedham, Westwood, and Norwood and southeasterly to Milton and Canton.
5. A 7.5-mil gal covered reservoir in the Blue Hills Reservation.

All of these proposed improvements were planned for construction in stages when and as the various towns sought admission to the Metropolitan District. Mains were sized to accommodate the anticipated flows in the year 2,000. In order to supply this greatly expanded Southern Extra High Service System, it was found that it would be necessary to extend the City Pressure Tunnel from Shaft 7B to a new shaft in the general vicinity of Franklin Field.

To date, the suction main and pumping station have been constructed and placed in service. The new Bellevue Reservoir is in the final stages of construction. Norwood has recently joined the Metropolitan District, and the 36-inch supply main south from Bellevue is under construction.

The force main from the Newton Street Pumping Station to the Bellevue Reservoir, which will enable the new station to supply the expanded Southern Extra High Service System, is in final stages of design.

Pumping Station—General

The pumping station (Fig. 1) is located on Newton Street in Brookline, on a lot formerly owned by the town of Brookline, and is adjacent to Brookline's old steam station, which is no longer used. Design of the new station was begun in June, 1953, by Coffin & Richardson, and construction was completed in April, 1955.

Considerable study was made regarding the type of power to be used for pump drive.

Diesel engines were selected for two reasons:

1. To eliminate the possibility of any interruption of the water supply to seven cities and towns such as would occur with electric motor-driven pumps in event of a power failure.
2. For economy of operation. A study indicated that the use of Diesel engines would result in an estimated annual saving of approximately $3,500.00.

The design incorporated all possible features to assure optimum noise abatement and elimination of vibration.

Prior to advertising for the general contract, the Metropolitan District Commission received bids on four requisitions for furnishing and delivering the necessary equipment. These were subdivided as follows:

1. Three Diesel engine-driven pumping units, including all auxiliary equipment.
2. All cast-iron pipe and fittings for installation both inside and outside of the station.
3. A complete instrumentation system, including a central control panel, and
4. Gate valves and check valves.

FIG. 1.

This procedure of purchasing the major items of equipment was followed in an effort to obtain better quality control. We also feel it contributed to a substantial saving, as we were able to design the building to accommodate specific equipment for which we had certified drawings.

Equipment

The initial installation includes three identical units, each consisting of a horizontal centrifugal pump, operating at approximately 1,470 rpm and driven through a 2.458-to-1 speed increaser by a 175-hp Diesel engine, operating at approximately 600 rpm (see Fig. 2). The engines are 6-cylinder, heavy-duty, 2-cycle, water-cooled, full Diesel engines, having a piston speed of 900 ft per min. All three units utilize a single compressed-air starting system (see Fig. 3) and operate on No. 2 fuel oil. The pumps and Diesel engines were manufactured by Fairbanks-Morse and Co., and the speed increaser by Philadelphia Gear Works.

Each unit is rated for pumping to the new Bellevue Reservoir at 5.10 mgd (3,540 gpm) against a total dynamic head of 134 ft. At this condition the pumps have an efficiency slightly in excess of 85%.

The cast-iron pipe and fittings within the property total in excess of 130 tons and vary in size from 12-inch to 30-inch. The U.S. Pipe and Foundry Co. were successful bidders for this item.

Hydraulic instrumentation at the station consists of a 30-inch Venturi tube for the total station discharge and a secondary 16-inch tube for metering the flow to Brookline. A central control panel (Fig. 4) contains a register-indicator-recorder for each Venturi, suction and discharge pressure indicators and recorders, and telemeter receivers to indicate and record the water elevation in the Bellevue and Brookline Reservoirs. An electrically operated indicating and recording tachometer for each pump is also included. The panel, measuring 7 ft high by 15 ft long, is of the dead-front type, and all instruments are accessible from the rear by means of a special instrument room. Builders-Providence, Inc., furnished the panel, Venturi tubes and accessory equipment.

Gate valves and check valves are of standard design and were supplied by the Chapman Valve and Manufacturing Co.

Felix J. Nesikian

FIG. 2.

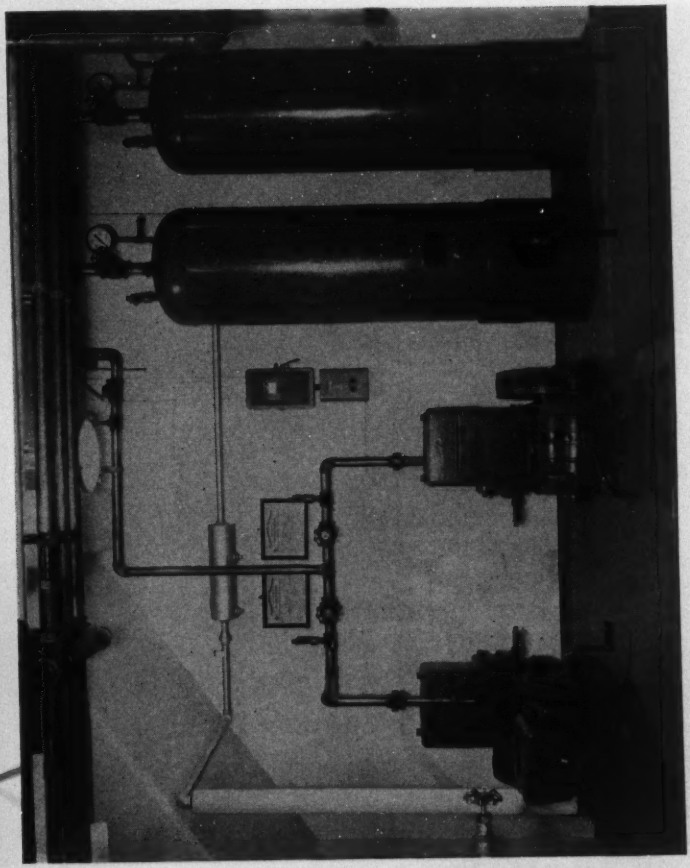

FIG. 3.

FIG. 4.

Building

The building consists of two principal areas. The front area contains 1,268 sq ft, consisting of an office, shop, locker-shower room, store room, and instrument-access room. There is no basement under this area. The pump room, which has a basement, extends toward the rear of the lot and contains 2,128 sq ft. Space was included for the addition of a fourth unit in the present pump room. The rear wall of this wing was designed so that an additional pump room can be added in the future, as the various cities and towns enter the District, and as the water-supply demands increase. Every effort was made in the design to locate all of the auxiliary equipment in the basement and to minimize the piping and equipment visible on the pump-room operating floor.

The building is a steel-framed structure. Exterior walls are Weymouth seam-face granite and the interior walls are glazed and acoustic tile. The pump room has a suspended acoustic tile ceiling. Floors throughout are red quarry tile.

The station was constructed by Sawyer Associates Construction Co., Inc., of Somerville, Mass., as general contractor. R. J. Berke & Co. of Newton was sub-contractor for the installation of all equipment.

The entire plant, including all equipment, cost $364,626.47. Acceptance tests indicated that the fuel cost per million gallons of water pumped will approximate $4.06.

The station was accepted by Harold J. Toole, Director and Chief Water Supply Engineer of the Metropolitan District Commission, Water Division, on April 15, 1955, and was placed in service on May 2, 1955, under the direction of Harry P. Morrissey, Superintendent of Pumping Stations. At present the station is supplying only a portion of Brookline, but upon completion of the 36-inch force main to Bellevue will begin pumping to the entire Southern Extra High Service System.

We at Coffin & Richardson have experienced a great satisfaction from being associated with this entire project and wish to take this opportunity to thank all those at the Metropolitan District Commission, the numerous manufacturers and suppliers, the contractor, and all of the sub-contractors who were engaged on the project. They all took a particular pride in their work, and we feel that their efforts are very evident in the completed structure.

THE HISTORY OF WATER ANALYSIS IN VERMONT

BY WILLARD B. FARNHAM*

[*Read at Chemists' Round Table, March 15, 1956.*]

In attempting to discuss the history of water analysis in my home state, I am handicapped by the fact that there are undoubtedly many of you here who know as much as, or more of, this subject than I, since I was not born until 17 years after the laboratory was founded and did not know of the existence of such a place until 1945, two weeks after my predecessor had retired. I was reasonably sure, however, that with a little diligent application I could develop a well-rounded story of the flowering of water testing from its first crude beginnings to the present scientifically exact determinations. Unfortunately, from my point of view at least, such was definitely not the case. Far from having any such crude beginnings, the theory and practice of sanitary water analysis were well developed at the time the laboratory was founded and have remained relatively unchanged to the present day. For practical purposes, therefore, but certainly not from any disinclination on my part, most of what follows is in the nature of a biographical sketch of my predecessor, the only previous sanitary chemist on the staff of the Board of Health—Charles Perkins Moat.

The Vermont Board of Health was established in 1886 after some years of agitation, although the collection of vital statistics had started some thirty years previously as a function of the Secretary of State's office.

In 1898, the twelve-year-old Board of Health, feeling the need for a laboratory, contracted with Dr. Jo M. Linsley at the University of Vermont to perform certain bacteriological tests, and in 1899 a laboratory was approved and an appropriation granted by the Vermont Legislature. Dr. Linsley was appointed director, and two other doctors, both of whom later became directors, were employed as assistants. An interesting sidelight of conditions at that time was the financial requirements set up by the legislature. The salary of the

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director could be set by the Board of Health up to $3,000, but the combined salaries of both assistants could not exceed $2,700. The total appropriation for the Board was $8,500 per annum, which included per-diem of $5 per day and expenses for Board members when attending meetings. At the time I joined the department in 1945, per-diem for Board members was $4 per day and expenses.

Water analysis was started on a routine basis immediately, both bacteriological and chemical. I found reference to previous analyses performed at the University of Vermont and Middlebury College, as well as analyses performed by outside consulting laboratories, costing $60 to $150. Many of the initial procedures were taken from Massachusetts, where analysis started in 1887, or Connecticut, where they started in 1889. Tests made at the time included sediment, turbidity, color, hot and cold odor, fixed volatile and total solids, hardness, albuminoid nitrogen, free ammonia, nitrites, nitrates, and chlorides, as well as total plate count and bacteria coli communis. Standards for the chemical tests were very smiliar to those used today. For the bacteriological they were somewhat higher than modern standards, since one-milliliter portions were used, but if we take into account the extreme sensitivity of the test, they were certainly sufficient to eliminate all sewage-polluted waters.

In 1901, after a short apprenticeship at the Massachusetts laboratories, C. P. Moat was employed as a sanitary chemist, the first and for many years almost the only non-medical man on the staff. An 1896 graduate of M.I.T., Moat was certainly one of the pioneers in the water field. Unfortunately, if he had any ambitions of making a reputation purely in chemistry, the situation in Vermont decisively eliminated them. Shortly after Moat was employed, Dr. Linsley resigned and shortly died. Dr. M. J. Wiltse was appointed director; Dr. B. H. Stone remained as bacteriologist, and Moat completed a three-man staff of the only functioning full-time division of the Department of Health. Even the secretary operated on a part-time basis from his home, so that the small laboratory staff was overburdened with administrative details while their technical work was increasing by leaps and bounds. Water samples, less than 500 in 1899, were over 1,000 by 1902. In addition, a great deal of field work was required and had to be done in a day when private transportation other than horse

and buggy was unknown. The people of the state and the legislature continually added new categories to the work done at the laboratory—food testing and medico-legal work, to mention two which affected the chemist. It was 1904 before any additional help was employed. At that time, a part-time medico-legal chemist and a bacteriological technician were hired. It was 1908 before another full-time chemist was employed and 1909 before a part-time sanitary engineer was available for part of the field work. It was not until 1939, 38 years after employment—when he had reached the age of 66, that a full-time sanitary engineer finally relieved Moat of the necessity of long and frequently tiring field trips during all of the year except the summer vacation period.

During this time the work load in the water-testing field continued to increase. From the level of about 750 in 1910 it jumped to more than 1,000 samples after World War I, to 3,000 by 1930, and 4,000 by 1935. During much of this period only one type of container was submitted—the one-gallon bottle on which a complete analysis was made. It was not until 1936 that a differentiation was made in the records between chemical and bacteriological samples, at which time about 1,500 chemical tests were made. From remarks made in the records and the manner in which the statistics were compiled, I am convinced that prior to 1930 nearly all of the 2,000-3,000 samples tested were chemical as well as bacteriological tests—a tremendous load for one man without assistance, who was also required much of the time to do milk, food, and occasionally medico-legal chemistry, to make frequent and extended field trips, and perhaps occasionally to attend a gathering such as this. He retired in 1945 at the age of 72, after 44 years' service, the only sanitary chemist ever to be employed at the Laboratory of Hygiene in the almost half-century of its existence. His salary, $1,200 when employed, had risen to slightly over $3,000 at retirement—certainly not a munificent salary for a lifetime of service. During his long term of service the laboratory occupied three different locations. Starting at 198 Main Street, it moved in 1911 to 184 Church Street, and in 1918 to 2 Colchester Avenue, all in Burlington. In 1953 we moved to a new building at 115 Colchester Avenue.

In connection with the above-mentioned preoccupation with medical men a peculiar incident occurred in 1921 on the death of Dr.

B. H. Stone, who had assumed the directorship on the resignation of Dr. Wiltse in 1904. Moat, by far the senior employee of the laboratory and a man well suited by temperament and ability to the position, was elevated to director, but after two months the Board of Health decided that a medical man should be director, demoted Moat to assistant director and promoted Dr. C. F. Whitney. I think it is a remarkable tribute to Moat that he continued to work faithfully for Dr. Whitney for more than 20 years, in spite of what must have been a bitter disappointment. In contrast to the early period there is today not a single M.D. in our laboratory staff.

Moat's contributions to public health in Vermont were many and varied. His first great project was to enumerate the public water supplies, and he almost immediately defined a public supply as one serving ten or more families—a definition still in use. By 1904 he had established a program of testing each public supply at least four times each year and continued that program until his retirement. He was the one most concerned in various Board of Health actions to force certain municipalities to change from a polluted supply to a safe one. In 1907 the Board of Health petitioned the Court of Chancery to enjoin the village of St. Johnsbury from using polluted Passumpsic River water for drinking purposes. The case, although won by the state in lower court, was remanded by the Supreme Court for technical reasons. However, St. Johnsbury and all the other towns saw the light, and it was necessary to go to court in only one other case, to force a municipality to cease using a polluted supply. Since the question of legal responsibility in this field had not been well established in 1907, this must surely be one of the precedent-setting cases in this field.

As the state enlarged its public health activities, Moat continued to be on the active front. In 1906 he instituted a program of lead testing, since lead pipe is common in certain parts of Vermont. Lead concentrations as high as 7.4 ppm were found in various supplies, certainly a toxic concentration if 0.1 ppm is considered the highest safe limit.

With the advent of regular inspection by a sanitary engineer, the water supplies of tourist houses and summer camps were tested routinely beginning soon after 1910. The railroad watering-point sup-

plies were tested for the U.S. Public Health Service beginning in 1914. All of these programs, while contributing to the public health program, also added to the work load. In addition, of course, it was necessary to keep abreast of important technical changes. In 1907 chloride of lime was recommended for the disinfection of sewage, and in 1910 for water, and soon after began to be used in Vermont. Rutland in 1917 installed the first liquid chlorine equipment to replace the hypochlorite previously used. During both wars the requirement of water-supply protection greatly increased the work, and in the nineteen-thirties the laws requiring licensing of bakeries and tourist houses, slaughterhouses and juvenile camps were passed, all of which required a water analysis before getting a license. By 1945 the volume of water samples was approaching 10,000 a year—a level at which it remains today.

When I replaced Moat in 1945 the program was essentially as I have described it. The staff, plagued by war-time shortages, consisted of the Director and medico-legal chemist, one sanitary chemist, one bacteriologist, one serologist, one or more technicians, and office help. Water samples were in the neighborhood of 7,000 a year, of which about 2,000 included a chemical test. The sanitary chemist was in nominal charge of sanitary bacteriology, although not doing all of the work. One technician was assigned to chemical water testing when available. Inasmuch as I was completely without experience, having started about two weeks after Moat's retirement, it took me some time to get acclimated.

My first recommendation was to adopt the U.S. Public Health Service standard bacteriological test for all public water supplies. This was accomplished shortly and has since been extended to all water samples. In the chemical end no important changes have been accomplished. We reduced the sample size from one gallon to two quarts, but have been unsuccessful in further reducing our requirements. In common with most laboratories we replaced the soap test by the versenate hardness titration. Moat had changed the total solids test to a pH determination in the thirties, and we simply changed from a colorimetric to an instrumental measurement. Other routine tests have been changed to instrumental methods, mostly colorimetric, including nitrite, nitrate, and lead. In nitrites we were successful in modifying our procedure so that a 10-ml sample is sufficient. In

addition we have expanded our facilities to allow more frequent testing for copper, iron, calcium, magnesium, fluorides, and manganese. Our staff has been expanded to include three full-time chemists, and we are now well equipped with instruments. Fundamentally, we are still performing the old-fashioned sanitary analysis. The methods we use are in general the standard methods outlined in "Standard Methods for the Examination of Water and Sewage". A reorganization in 1951 placed all bacteriological work under the chief bacteriologist, so we are now concerned only with chemistry. In addition, the emphasis of the water-testing program has been placed on the more sensitive bacteriological test, and we are now doing less than 500 routine chemical tests a year.

I think you will agree with me that the above rather brief description of our present status represents little improvement in program over 1900. However unflattering the picture may be, we don't really intend to throw in the towel. For the past two years as time allowed we have been setting up a complete mineral-testing program along modern lines. The one determination that bothered us was a sulfate determination. We didn't really want to adopt a gravimetric determination for a routine method because of our limited staff, but nothing else seemed accurate enough.

Recently, however, one of our chemists has developed the turbidimetric method in "Standard Methods" with fairly good results. Right now I'm optimistic but not entirely convinced. The trouble has been that our normal sulfate concentrations are so low that even gravimetric methods have considerable error. The other ions to be determined are fairly well worked out, including calcium, magnesium, iron, manganese, fluoride, chlorides, nitrate, total alkalinity, and sodium and potassium by flame photometry. We have a Beckman D.U. Spectrophotometer with flame attachment. We feel that within a year we should be in a position to perform a routine mineral analysis or a routine sanitary analysis, or both if desired.

There is one more thing which I'd like to mention before I close. I have not said much about sewage analysis. I realize that this isn't primarily a sewage orgnization, but I'm sure some of you are concerned with sewage and many public health departments are concerned with both water supply and sewage. In Vermont prior to 1949

the Vermont Department of Health was doing what sewage work was being done. However, the 1949 legislature, in passing the "Sewage Treatment Plant Enabling Act", placed responsibility for enforcement of stream pollution laws in the Water Conservation Board, which prior to 1949 was a flood control commission. As a result, the Water Conservation Board has set up laboratory facilities, including a mobile laboratory, to perform these water tests. With the exception of this and the laboratory tests performed by the city of Burlington water and sewage departments, all public water testing for Vermont is done at the Vermont Department of Health, Bureau of Laboratories.

WATER FOR WARTIME FIRE PROTECTION

BY HORATIO BOND*

[*Read March 15, 1956.*]

What may be said about wartime problems in providing water for public fire protection also has peacetime applications. Usual water supplies have often been interrupted by earthquakes, storms, floods, landslides and various other catastrophes. Pumping stations can be destroyed by fire.

There are perhaps four divisions under which this subject can be conveniently considered, and I would state them about as follows:

1. Alternatives in design of public water systems for fire protection.
2. Measures which may be taken with our present water utilities to offset damage to all or part of the system by bombings or by other catastrophes.
3. Provision of portable supplies of water by fire departments.
4. Development of auxiliary water supplies, principally from natural bodies of water or by means of storage facilities.

Design Alternatives

Traditionally, water has been supplied for public fire protection in a number of ways. San Francisco, for example, still has underground cisterns, and water holes are still used in many rural and farm areas. It became early apparent that the most convenient and economical arrangement was a single public water system, providing the amounts of water required both for fire-protection purposes and other uses. Before the days of the automobile pumping engine, a number of large cities also installed so-called high-pressure systems, which were separate systems of street mains, to provide a water supply of superior capacity and reliability in high-value areas. No new systems of this sort have been installed for many years, and the special pumps for systems in New York City have actually been discontinued, be-

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cause adequate pressures are now obtainable from fire-department automobile pumpers.

I believe that it is time for cities to ask their engineering advisors again to take a look at the fundamental approach to the design of municipal water supplies for fire protection, in the light of conditions which may exist in wartime. Some of you will remember that in 1943 I presented a paper on "Water Supplies for Air Raid Fires" before the New England Water Works Association meeting in Boston. This was published in the JOURNAL of the NEWWA, March, 1944, vol. 58, no. 1, page 36. The paper was a report on English experience with emergency water supplies. In it I offered a suggestion which might help to solve the problem of damage to fire-protection facilities. This is the only suggestion which has appeared, as far as I know, in the water-works literature, suggesting any alternative to conventional design.

It suggested a separate system of street mains carrying water for fire purposes only. The principle would be the same as the separate fire-main systems which numerous cities now have, but the arrangement of piping would be different. Instead of a single system covering an entire section of the city, there would be a number of relatively small systems, each normally independent of the adjoining one. Such systems would cover large single blocks or three or four small blocks. The pipe layout would be much the same as for systems of separate fire mains in the present practice of protecting large industrial plants. Each system would have a gravity tank and a ground storage tank. The purpose of the ground storage tank would be to provide the total amount of water needed, and the tank would be located at some convenient location in the block or group of blocks. In a city so protected, there could be extensive damage, but in any section where buildings remained standing their water system for fire protection would probably be intact.

Breaks in Existing Systems

A water utility may usually be counted on to have in force procedures for dealing with interruptions of service. In case of a wartime attack on a city, wall failures in buildings could be expected to break thousands of the water connections and cause loss of water in the

street system. The extent to which loss of water can be avoided depends on how promptly valves between sections of the system can be operated to stop waste. However, the operation of any appreciable number of valves requires numbers of experienced water-department employees and equipment, which may not be available. Fire-fighting plans have to be based on the assumption that interruptions of the public water system may be of several hours' duration in the case of any one break in a distributing main, and several days' duration in the case of main feeders. The water utility will need written agreements to define the terms under which, in emergency, one may get water from systems of adjoining communities. Each utility should also study measures to increase the reliability of the water supply, such as storage tanks and reservoirs.

Auxiliary Water Supplies

There are also possibilities of developing auxiliary water supplies, using street tanks and bodies of natural water. In 1952 the Massachusetts Civil Defense Agency had an engineering study made, to suggest a method of developing wartime water supplies for fire fighting in a part of the Boston metropolitan district. These engineering proposals were practical and sound, and showed the sort of thing involved in developing auxiliary water supplies.

* * * *

I have covered very sketchily the considerable ramifications of this subject. I hope that at this session instructive thoughts will be offered as to the most fruitful lines of work to be pursued in reaching solutions for the problem of water supplies for wartime fire protection.†

†See this JOURNAL, 70:3, pp. 311 and 327.

CONSTRUCTION OF MIANUS RIVER DAM AND RESERVOIR FOR GREENWICH WATER COMPANY

BY GEORGE E. PAUL*

[*Read April 19, 1956.*]

In our study for a reservoir on the Mianus River it was found that there were no maps which showed sufficient detail and information for our needs. Therefore, on December 5, 1950, several aerial survey flights were made, to obtain strings of photographs from which could be made an accurate topographical contour map. By means of this map, together with reconnaissance trips on the ground, a suitable site for the dam was selected.

The drainage area of the Mianus River lies largely in the southwest corner of the State of Connecticut, with a considerable portion extending into the State of New York. There is a total catchment area of 30.75 sq mi from the headwaters to the mouth in Long Island Sound. There are 18.1 sq mi of drainage area above the dam site.

The stream has its headwaters in Bankville, Conn. It then flows into New York State, where it meanders considerably, and then again enters Connecticut and flows south 1.75 mi below the state line to the dam site. From the dam site to Long Island Sound is about 8 mi. The total length of the river is about 20.4 mi.

The topography of the drainage area above the dam is generally rolling or rugged ground with occasional hills 300 ft or more above the river bed. At the upper end of the reservoir the river flows through a deep gorge, known locally as the "Mianus Gorge". It is a favorite haunt of nature lovers. The topography of the area has been formed by the dissection of a plateau and the deposition of glacial drift over the surface. Hilltops are generally boulder-strewn grasslands, suitable only as pasture lands. The stream bed near the dam site was only 10 to 15 ft wide. There were no highways, railroads or permanent structures in the reservoir area. Below the dam is a more developed, exclusive residential area.

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The geology of the Mianus River basin is similar to the geology of the State of Connecticut. During the glacial period the Labrador ice sheet moved south, scraping off the soils and rock in its path, and deposited this miscellaneous mixture of sand, clay, pebbles and boulders, weighing many tons, upon the landscape. The underlying rock is schist or gneiss. Outcropping of rock is common at and near the dam site.

We had a 68-year record of rainfall for the area, which showed an annual average rainfall of 43.4 in. The minimum of 29.9 in. occurred in 1935 and the maximum of 59.1 in. in 1951. Percolation of rainfall into underground strata is estimated at 20% of rainfall. The average air temperature is 49.5° F.

We found that the low stream flow was 660,000 gal per day. However, this flow is sufficient to maintain a minimum stream flow below the dam of 4 mgd from an impounded-supply reserve of only 250 mil gal. In using the State of Connecticut curves we found the largest flood from a drainage area of 18.1 sq mi would be 5,900 cfs.

Having satisfied ourselves that our preliminary plans and findings would warrant building a dam and reservoir, we proceeded with our final plans. The original river bed at the dam site was at elevation 187.00. Main spillway crest is at elevation 252.00, providing a storage of 2.2 billion gal and a lake area of 216 acres. We cleared land to elevation 257.00, a total area of more than 225 acres. The maximum wave with a wind velocity of 60 mph over a distance of 2.6 mi is only 3 ft. We established the top of dam at elevation 262.00, or 10 ft above spillway level.

A construction contract was awarded March 1, 1954, and impounding began May 23, 1955.

There were many preliminaries to actual construction of the dam. These included the construction of access roads into the area, the construction of a temporary earth diversion dike, and the installation of a 5-foot galvanized-steel conduit for use in diverting the existing stream during construction. This 5-foot temporary conduit joined the permanent 8-foot-diameter overflow conduit at a cross, directly below the vertical morning-glory spillway. All permanent pipelines through the embankment were bedded in concrete and also had steel collars at every joint, to prevent seepage of water along the pipeline through the dam. After the stream had been diverted through the

dam, work began in earnest on moving the large quantities of loam and unsatisfactory earth materials, which had to be stripped from the base area of the dam in order to obtain a good bond of the new materials against the underlying soils.

One of the early construction features was to provide the sand and gravel filter underneath the center of the dam. This was placed before earth embankment was started, as portions of the filter had to be depressed below original ground. Embankment followed shortly thereafter. Materials for the embankment were brought from borrow pits, the closest of which was approximately 1,000 ft away from the dam, ranging up to a borrow pit over a mile away. The very select clay which formed the impervious clay core-wall was trucked from our Rockwood Lake property, about 4½ mi away. At the height of construction the contractor moved about 10,000 cu yd per day.

We worked through the entire winter and there were very few days lost; however, it was necessary before starting work each morning to scrape off the frozen top layer of materials which had been placed the previous day, so that the new materials could be properly bonded and rolled into the dam. There was one period of very cold weather when the work stood idle for approximately a week. Subsequent to this period, in order to get started placing earth embankment again, it was necessary to blast the frozen materials in order to get down to an unfrozen base.

The entire dam consists of approximately 375,000 cu yd of materials. These include earth, clay, gravel filter, crushed-stone blanket and rock facing. At the base of the clay core-wall we had a trench, approximately 20 ft wide and 15 ft deep, to act as a water seal to prevent water from seeping along the original ground line and percolating through the base of the dam. We tested by core drilling in the bottom of the core-wall trench to determine whether there were voids below our structure. There were very few voids, although we were prepared to do pressure cement grouting. It was generally not necessary to grout.

The clearing of the land, which was in progress throughout most of the job, required that we transport about 40 timber men from the State of Maine, to clear the 225 acres of reservoir area which would be flooded. Many of these men lived in barracks right on the reservoir site. They had their own cooks and did their bathing in the Mianus

River, even in the coldest weather. Some of the timber which was cut was sawed into planks right on the site. It was found uneconomical to haul timber from the farthest ends of the reservoir; therefore, considerable burning took place as the trees were felled. Large bonfires were kept burning day and night until the disposition of all cuttings had been completed. A privately owned fire truck was kept at hand to control the fires.

On May 23, 1955, the work had been sufficiently completed so that we could remove the temporary 5-foot conduit and plug its opening into the dam. Following this we removed the temporary dike above the dam, and water began collecting at the upstream side of the dam. In view of unsettled matters relating to the interstate complex of the development, it was decided to restrict the impounded water temporarily to an area between the dam and the Connecticut state line. This was done until October 15, 1955. On the weekend of October 15th and 16th, you may recall, there were torrential rains throughout the New England area. These rains contributed to the waters above the Mianus Dam and, disregarding the state line, the waters rose 34 ft in two days and overflowed the morning-glory spillway at least 2 ft. We found that approximately 1.85 billion gal of water were stored in the reservoir in these two days. The dam immediately proved by flood that it was entirely adequate for the job it is to do for the coming years. And, incidentally, it saved the downstream property owners from extensive flood damage, which would have occurred had our dam not been there.

The dam consists of earth embankment, 20 ft wide at the top and 965 ft long. The upstream face slopes 3.5 horizontal to 1 vertical near the bottom and 2 to 1 at the top. The downstream face is the same. There is an impervious clay core throughout the height of the dam, beginning with an excavated trench filled with clay below original ground level (see Fig. 1). The remainder of the embankment is semi-pervious, compacted earth embankment. We used soil compaction at or near optimum moisture to dry densities of 125 to 140 lb per cu ft. Under the center of the dam at original ground line we provided a sand and gravel filter, so graded that ground water could filter into it from either top or bottom without disturbing or dislodging the earth embankment around it. Porous concrete pipes in the

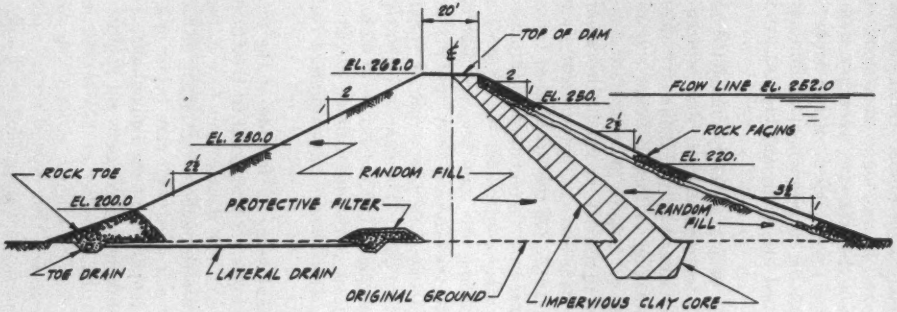

FIG. 1. SECTION THROUGH MIANUS RIVER DAM.

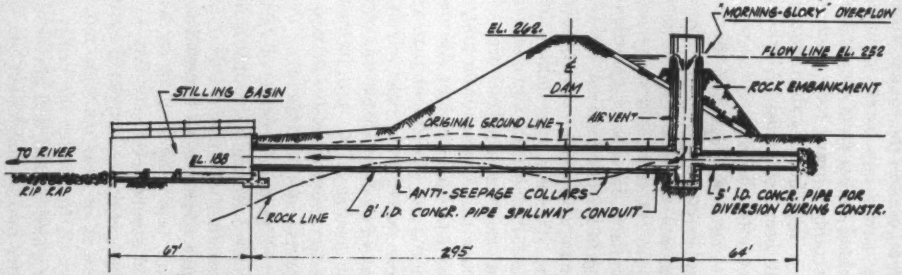

FIG. 2. SECTION THROUGH SPILLWAY CONDUIT—MIANUS RIVER DAM.

filter section collect any water present and then lead it by tightly jointed pipes at intervals to the downstream toe drain. There is also a toe drain filter and drain lateral all along the toe, to dispose of any water from ground sources. A heavy rock toe section was provided above the downstream toe drain, to stabilize the fill and prevent sliding of earth materials. This rock toe section also allows percolation of surface rain water into the buried drainage piping, which flows to the river below the dam.

This dam has a vertical, 8-foot-diameter morning-glory spillway with a 12-foot-diameter lip, which will pass 2,200 cfs. After dropping vertically 62 ft, the water continues through an 8-foot-diameter concrete conduit to the concrete stilling basin (see Fig. 2). The morning-glory spillway will pass all major floods except the 100-year flood. An auxiliary wasteway has been provided to take care of extraordinary floods. It will pass 4,400 cfs at a 7-ft depth of flow. The total capacity of both spillway and wasteway is 6,600 cfs, which is in excess of the State's requirements.

Adjacent to the stilling basin there is an underground concrete vault, housing two motor-driven pumps, which will be used to draw water from the reservoir through a 30-inch intake pipe, and pump it over the hill several miles to our Putnam Lake filter plant. Interconnecting piping at the dam also allows us to release water from the reservoir to Mianus River below the dam. Here it flows by stream to our Mianus River filter plant. We plan to regulate the flow by automatic control devices at the dam to pass a minimum of 4 mgd for use of downstream riparian owners. This will also provide the supply for the filter plant, which is 2 miles above the mouth of the river in Cos Cob Bay.

After allowing 10% of the total storage to remain in the reservoirs as a safety factor and allowing for evaporation, transpiration and other losses, as well as minimum release of 1.5 mgd at the Mianus filter plant, and assuming the right of diversion, the safe yield of the Greenwich Water Company's sources of supply in total is estimated to be 17.5 mgd.

NOTE: Since the reading of the above paper the Greenwich Water Co. has named the reservoir "The Samuel J. Bargh Reservoir", in tribute to the late Mr. Bargh, who was formerly New England Division Manager, American Water Works Service Co., Inc., and President of the Greenwich Water Co.

THE GREENWICH WATER COMPANY MIANUS FILTER PLANT

BY HOWARD J. CARLOCK*

[*Read April 19, 1956.*]

The Mianus Filter Plant is the latest addition to the treatment works of the Greenwich Water Company. This plant has a rated capacity of 4.0 mgd, with provisions for increasing the rated capacity to 12.0 mgd by the addition of four more purification units (see Fig. 1). Maximum capacity of the plant will be 18.0 mgd at a filter rate of 3 gal per sq ft per min, which represents a 50% overload of the rated capacity. The plant is located on the Mianus River, approximately 3¼ mi east of the main plant at Putnam Lake. The source of supply for this plant is the Mianus River, the flow of which is controlled by the outlet works of the Mianus River Dam, located 4½ mi northwest on the same river. The dam is of earth-fill construction with a maximum height of 75 ft and impounds 2.2 billion gallons of water. This storage reservoir and the filter plant provide the supplementary supply for the Greenwich water system.

Site Conditions

Selection of a site for the plant was influenced by the hydraulic conditions available. The site on which the plant was constructed was formerly occupied by a manufacturing mill, long since abandoned. The river was dammed at this location, and the dam was found to be in good condition. The dam offered the advantage of a hydraulic gradient sufficient to provide gravity flow through the treatment works and eliminate the necessity for low-service pumping. The disadvantages of the site were the preparatory work and expense to remove the existing foundations of the mill structures, and the limited area available. However, preliminary studies indicated that the maximum size of plant contemplated, based on the yield of the stream and the Water

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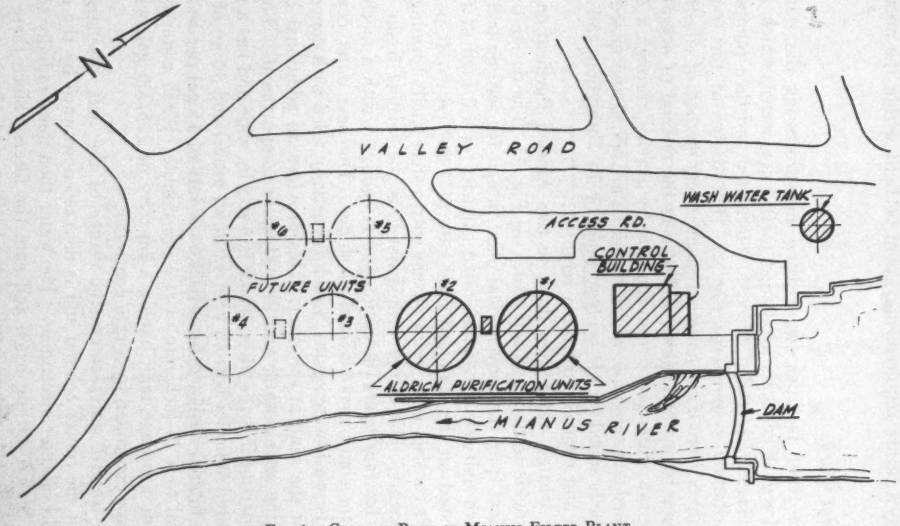

FIG. 1. GENERAL PLAN OF MIANUS FILTER PLANT.

Company's future requirements, could be built within the area available.

Treatment Works

The plant facilities for treating and processing the water are conventional. Chemicals are added to the raw water, which is then mixed, coagulated, settled, filtered, chlorinated and then pumped into the distribution system (see Fig. 2). The design of the treatment works differs from a conventional plant in the method of constructing the settling basins and filters, and also in the method of operation.

The raw water flows from the dam by gravity, through a 30-inch pre-stressed concrete pipe, where chemicals are added and mixed in the line. Lime is added for pH control, alum for floc formation, and carbon for taste and odor control when required.

The treated water then flows to two Aldrich Purification Units which provide settling and filtration (see Fig. 3). The design of these units differs from the conventional plant design for settling and filtration. Each unit has a nominal rated capacity of 2.0 mgd of filtered water, based on the standard filter rate of 2 gal per sq ft per min. Each unit consists of a series of concentric steel shells forming three compartments. The center compartment is the settling basin, which provides for four hours' detention and is equipped with a mechanical clarifier for continuous removal of sludge. The next outer compartment is the wash-water gullet, which is partially filled with concrete to a level of the sand in the filter. This provides an additional barrier between the settled-water and filtered-water compartments. The outside compartment is the filter, which is 3 ft 5 in. wide and has a monolithic poured Wheeler bottom with 12 in. of gravel and 30 in. of sand.

The filter is designed for a backwash rate of 15 gal per sq ft per min, equal to a 24-inch rate of rise per minute. Wash water is supplied from a storage tank having a capacity of 225,000 gal, with its rate of flow controlled by a rate controller.

The treated water enters the center of the unit, discharges at the top of the center column and is directed downward by the center well on the clarifier mechanism. The water then flows radially toward the shell of the settling basin and onto the filters. Since there is no connecting pipe or valves between the settling and filtering compart-

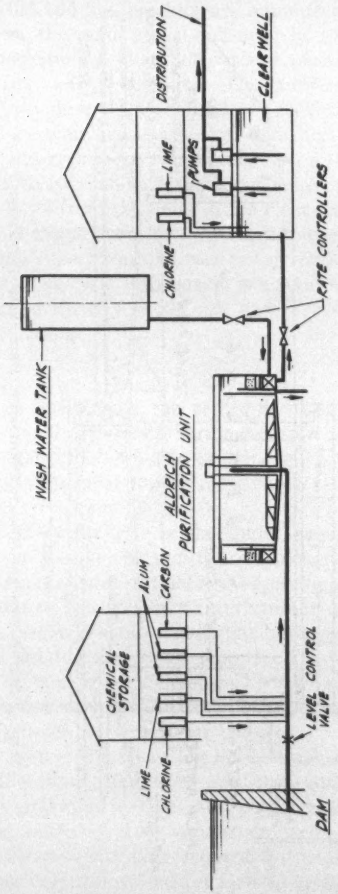

FIG. 2. FLOW DIAGRAM FOR MIANUS FILTER PLANT.

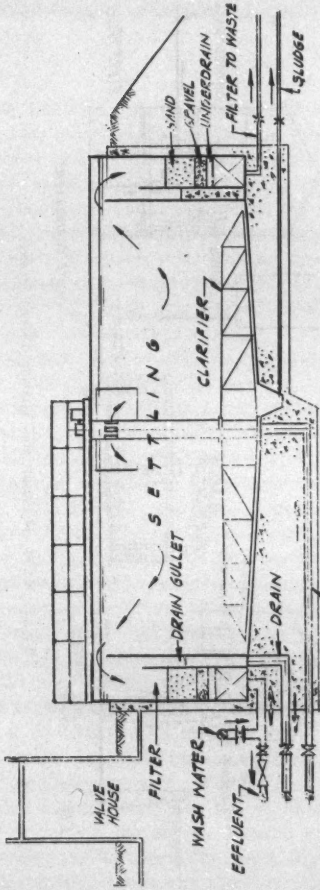

FIG. 3. SECTION THROUGH ALDRICH PURIFICATION UNIT AT MIANUS FILTER PLANT.

ments, loss of head and floc breakup are minimized. The water flows downward through the sand, gravel and filter bottom, and is collected in a chamber beneath the filter bottom and piped to the clear-well, located under the control building. The purification units are so designed as to provide a positive head in excess of eight feet above the sand. This permits head losses of eight feet through the sand and bottom without creating a partial vacuum on the underside of the filter bottom, which is common in most conventional designs.

The water flows from the units to the clear-water basin, where it is chlorinated and the pH corrected by the addition of lime. Vertical-turbine type pumps discharge the water from this basin into the distribution system. The clear-water basin is a reinforced-concrete structure, having a capacity of 138,000 gal, and serves as the foundation for the control building.

Control Building

The Control Building is the center of operations. It was designed with a colonial exterior to harmonize with the general area. It is a four-story structure with two stories above the finished grade. The lowest level is the clear-water basin, which is used as the suction well for the high-service pumps.

On top of the clear-well is the pump room. Two 1,400-gpm and one 2,800-gpm motor-driven units are provided for normal service. Standby equipment consists of one 2,800-gpm diesel-driven unit, with an extension shaft to provide for a future motor drive. An auxiliary, gasoline-driven, power generator unit, having a capacity of 25 KVA, is provided for lighting and small motors. The wash-water tank is supplied by a 700-gpm vertical pump. The wash-water piping is such that the main pumps can pump directly into the wash-water line when the storage tank is out of service for painting.

At ground level is the operating floor, which contains the chlorinators, chemical feeders, laboratory and filter controls. Two manual chlorinators are provided for pre- and post-chlorination, with provision for a third machine. Two volumetric feeders are provided for lime, and two submergence-type feeders are provided for alum. A volumetric feeder is also provided for feeding carbon. Space has been provided for adding additional feeders when required. The lime and

carbon feeders are provided with dust collectors, to help keep the building clean.

The top floor is for storage of chemicals and chlorine cylinders. Storage area is available for 82 tons of alum and lime with chemicals stacked on pallets two tiers high. Separate storage rooms are provided with outside entrances for carbon and chlorine. The carbon room has capacity for 4 tons and the chlorine room stores 60 containers of chlorine. Storage for lime, alum and chlorine is based on three months' requirements at maximum future filter capacity. Carbon is used very infrequently and storage was on the basis of a small truck load.

Operation

The method of operation was one of the prime factors influencing the design of this plant. Because of its location, which is removed from the main plant, it was necessary to provide additional labor for operation. To minimize this expense, it was decided to use automatic controls for operation and washing of the filters, so as to require only one operator per shift.

The purification units are controlled from the panel board located in the laboratory. These controls operate pneumatically to control hydraulic valves, located in the valve house between the two units. When loss of head in the filters reaches a predetermined level, an alarm rings and a light indicates the filter requiring washing. The operator then pushes a button, which starts a program timer. This timer actuates pneumatic relays, which open and close the various filter valves to start the wash cycle. An interlock is provided, so that each valve in the sequence must be fully opened or closed, as required, before the next valve functions. As the wash-water valve starts to open, the wash cycle goes into operation. Filters are washed in three stages. The initial wash is at a low rate, to raise the bed; the second cycle is the high rate, to clean the sand, and the third cycle is at a low rate, to settle the sand bed. The length of time desired for each stage of washing is adjustable at the control panel, as are the rates of wash. When the wash cycle is completed, the program timer completes the cycle by reversing the operation of the various filter valves.

Once the operator has pushed the button to start the program, he is no longer required in attendance at the control panel. If the opera-

tor wishes to observe the wash, he can walk out to the unit, where a manual operation station is located, to control the length of wash. This control interrupts the program, to increase the time of washing as required.

Level is maintained in the purification units by a butterfly valve in the raw-water line. When a purification unit is taken out of service, a pneumatic level control changes the air pressure on a valve positioner on the butterfly valve. This positioner controls the pressure water supply to a cylinder on the butterfly valve and closes the valve sufficiently to equalize the flow in and out of the purification unit still in service. When the first unit is put back in service after washing, the level in the second unit starts to drop, and the butterfly valve opens to permit more water to flow to the units. The level in the units is controlled within a six-inch range.

The control panel also contains remote rate setters for each filter, to change the filter rate as required. These rate setters control the effluent rate controllers, located in the valve house.

Manual hydraulic controls are provided in the valve house, to operate the hydraulic valves for each unit as an auxiliary method of operation. Rates of feed for chlorine and chemicals are adjusted manually. The chemical feeders and chlorinators are in rooms adjoining the laboratory where the filter controls are located, thus making it convenient for the operator to control the treatment.

During 1955 the plant averaged 3.5 mgd output. The filters used 2.8% wash water, with an average filter run of 36 hours and maximum runs of 55 hours. The average cost of water produced at this plant was $16.24 per mil gal, including labor, chemicals, fuel and power.

Color removal is the chief factor to contend with. The raw water averages 45 ppm of color, with a maximum of more than 100 ppm. Treatment reduces the color to an average of 2.0 ppm in the effluent.

General

As a private utility, capital expenditures are of prime concern in the construction of a new plant. The Mianus Filter Plant cost $532,000, exclusive of land and overheads. This is equal to $133,000 per mil gal of filter capacity. It was estimated that if the same type of plant were constructed with conventional settling basins and con-

crete filters, including housing and interconnecting piping and valves, the plant would cost $720,000, or $180,000 per mil gal. Additional filter and settling capacity can be added to the present plant by the addition of purification units and connecting piping. It is estimated that the future additions will cost $50,000 per mil gal of filter capacity and will include settling, filtration, piping, valves and automatic controls.

The purification unit design was developed by E. H. Aldrich, Chief Engineer of American Water Works Service Co., Inc., for the purpose of providing the same quality of finished water as conventional plants, but at a much lower initial cost. He pioneered in the application of automatic controls for filter-plant operation as a means of reducing plant operating costs.

This type of plant has been approved in 12 states by the various health departments, and the American Water Works Service Co. has 14 installations in service, of which nine are automatically controlled. Of the 14 installations, 12 provide standard treatment, one is used for softening, and one for iron removal in a well supply.

The first plant of this type was constructed in Alexandria, Va., in 1950 at a rated capacity of 9.0 mgd. Additional units were added in 1953 and 1955, to increase the capacity to 18.0 mgd.

The purification units are marketed by the Dorr-Oliver Co. as Dorco-Aldrich Perifilters. The automatic filter-control system was furnished for this plant by Builders-Providence, Inc., and is known as their Auto-Central Control. This control system has been used successfully at our eight other automatically controlled installations.

Aldrich Purification Units are adaptable to the treatment of most waters when settling and filtration are required. Other mechanical devices can be used in the settling compartment, such as blanket-type upflow units, flocculation and aeration equipment. The degree of automatic control can be increased or decreased to satisfy the requirements of the process.

WATER WORKS AT GENEVA, SWITZERLAND

BY E. SHERMAN CHASE*

[*Received April 21, 1956.*]

At the time of my visit to Geneva in 1955 I called at the office of the water-works department, of which the director is M. Paul Pazziani, a member of the New England Water Works Association since 1935. In addition to a pleasant chat with M. Pazziani, I was conducted to various parts of the Geneva water works by two members of his staff, Messrs. Georges Raymond and Herbert Pettman.

The water supply of Geneva is derived mainly from Lake Geneva, untreated except for chlorination. Part of the supply comes from deep wells. A description of the water works by M. A. Betant, former director, can be found in the JOURNAL of this Association for March, 1921.

Due to algae troubles and to increasing hazard of pollution, a modern filtration plant is now under construction, to be completed in 1958. The first stage of this project consisted of laying a new subaqueous intake pipeline into the lake. A description of this work is given in a "Bulletin of the Geneva Utilities" (No. 2, 1953). Miss M. S. Little, technical writer of Metcalf & Eddy, has prepared the following translation of the description given in the Bulletin, which is presented here with M. Pazziani's permission.

LAUNCHING, TOWING AND LAYING OF THE SUBAQUEOUS INTAKE PIPELINE

By P. Peitrequin, Civil Engineer†

The carrying out of the operations of the construction project with which this paper deals constituted one of the most difficult problems in the art of engineering. As far as we know, there is no other project of such an extent anywhere in the world. If the Americans

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†One of two engineers directing the work.

had conceived a "pipeline" of such a diameter, the technical literature would have revealed it. To be sure, there was the memorable pipeline crossing the British Channel, which supplied motor fuel to the Allied invasion troops, but the process of laying it was apparently like the one used with submarine cable, rather than the method that we are going to describe.

Extraordinary piece of work, we call it, both on account of its dimensions—length, 3,000 meters (9,843 ft), diameter 1.6 meters (5-1/4 ft), total weight 1,000 tons—and on account of the special conditions of its rigging, towing, and laying in place at the bottom of the lake, as far down as 40 meters (over 131 ft).

The site chosen at Geneva for laying the pipeline required the dredging of a channel for a certain distance out into the lake, designed to receive and protect the pipeline in the area where the water is too shallow. The huge floating dredge employed was constantly guided in its operations by a team of surveyors, stationed on the shore. Their orders were transmitted over portable radio-telephones (walkie-talkies).

A few meters (yards) away from Wilson Wharf a concrete anchorage block was constructed, protected by a fencing of metallic sheet piling. This permanent structure is pierced by three pipelines. The largest in diameter, 1.6 meters (5-1/4 ft), forms the primary element of the intake pipeline under discussion. The second, 1.4 meters (4-3/5 ft) in diameter, constitutes the point of departure into the lake of the filtered-water pipeline, running from the filtration plant and ending in the pumping station at Coulouvrenière. Finally a third pipe, with a diameter of 0.9 meter (nearly 3 ft), serves as the outlet for the wash-water from the filters.

Three kilometers of pipeline can scarcely be laid down as a single-unit operation. This length was divided off into 6 sections, the first one 280 meters long (919 ft) up to an elbow joint, and the others each 540 meters long (1,772 ft). After study of the entire shoreline of the Lake of Geneva, the beach at Préverenges, east of Morges, was chosen for the rigging operations, and the length of the straightaway part of this beach was our guide in determining the maximum length of the pipe sections.

The pipes were shipped by truck from the foundry (located be-

tween Lausanne and Renens), in units 10 meters long (about 33 ft), with each unit weighing 3 tons. Unloaded at Préverenges by gantries and block-and-tackle gear, these pipes were next placed on narrow-gage railway cars, running back and forth along the beach, hauled back and forth by winch. Each unit was rolled sideways onto a wooden trestle, the pipes being placed end to end in this way and delivered to the welders, who welded the joints together, inside and out. The finished section was slid sideways onto the previously constructed launching ramps (iron frames on concrete footings), which were arranged perpendicular to the shore, and sloped down into the water. Taking into account the resistance of the pipe, these ramps could have been as long as 80 meters (262 ft), but a length of only 40 meters (131 ft) was decided on, to make the launching easier. In this manner, the six sections were placed successively in parallel, side by side on these ramps.

The pipes were not smooth on the outside: what were the various annular elements? On the first two sections there were reinforcing hoops, which were not necessary in the later sections; then the joints, which allowed the pipeline to make angles, special watertight members in the shape of flanges 2.4 meters (7.9 ft) in outside diameter (see Fig. 1); finally, pairs of rings to which the "floats" could be attached.

What were these floats? During the operation of laying the pipeline on the bottom of the lake, the weight of one large section was 140 metric tons (154 U.S. tons), under water. To support this load solely by floating barges would have required a considerable fleet. So the idea was conceived of lightening this weight by attaching buoyant floats to the pipeline, rigidly fastened to it and gliding with it. These floats were gaged with exactness and were distributed along the section so carefully that the barges floating on the lake had to carry by means of their lowering winches only a small fraction of the total load. When the pipeline came to rest on the lake-bed, these floats were detached by the diver after he had bolted the sections together, and then they were hauled to the surface and towed back to Préverenges, to be fastened to the next section.

At Préverenges the sections rested on launching ramps that sloped down to the ground, and the question was how to get the sections afloat. Since the water wasn't deep enough for the floating

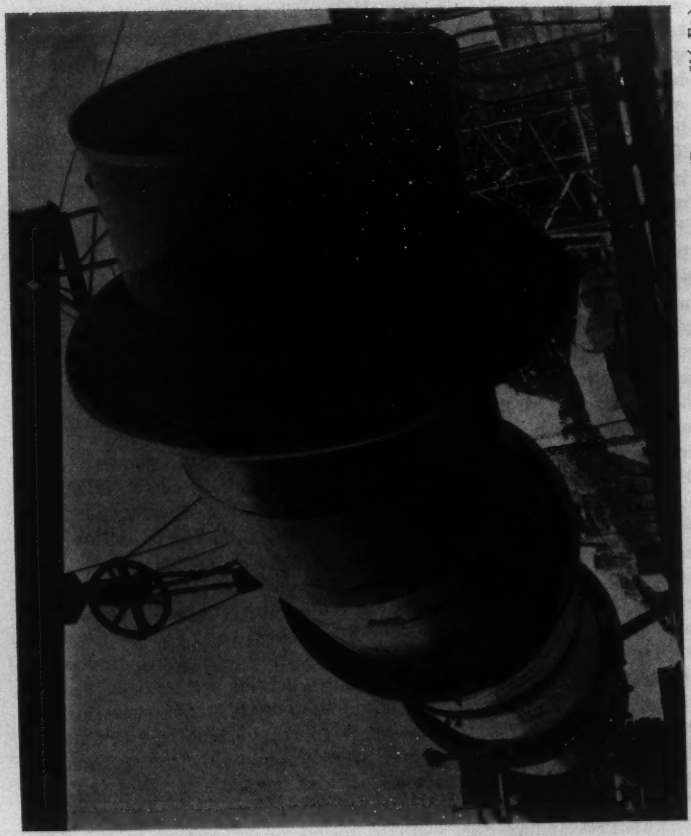

FIG. 1. ASSEMBLY OF THE INTAKE PIPE OF THE SUBAQUEOUS RAW-WATER CONDUIT (DIAMETER, 5¼ FT.)

operation, a sizable channel was dug at the foot of the ramps by mechanical shovels and a "bulldozer." This channel, about 800 meters long (2,625 ft), joined the mouth of the Venoge River, from which vicinity the sections started on their way. Each section, covered at both ends with watertight shields, was slid into the water and towed along the channel. It was here that the floats were attached. Gantries spanned the channel at its entrance into the lake. The floats, which consisted of two tanks joined in pairs, were hoisted by tackle operating from the gantries and could be set in place over the pipeline like a saddle, as the towing proceeded.

The pipeline section, equipped with its eight floats, was taken in tow by the vessel *"Geneva"* of the C.G.N. (see Fig. 2), which, after a wide swing, steered toward Geneva. The voyage of 40 kilometers (25 miles) as the crow flies, was sailed in 4 to 5 hours (see Fig. 3).

While the launching was proceeding at Préverenges, the equipment at Geneva was not resting idle. Concrete abutments had been anchored on each side of the route, with cables terminating in a mooring buoy. There were moorings fore and aft of the metal barges, which were placed squarely straddling the route of the section to be laid. These barges, with 11 allotted to each 540-meter section (1,772 ft), were each equipped with a mast crane and four winches, one the principal lowering winch, the others the mooring winches. The positioning of the barges was given by walkie-talkies, according to plan, from the shore where the surveyors were stationed with theodolites.

The pipeline section arrived alongside the first barge; the *"Geneva"* handed over its towline to a tugboat which guided the pipeline in front of the barges, beneath their cranes. The suspension operation from the hooks of the main winches was accomplished, and from this moment the section was at right angles to its final position, protected from the danger of breaking loose, although it could still float by itself (see Fig. 4). A pump coupled to one end filled the pipe section in 2 hours. To immerse it completely, 1.1 cubic meters (39 cu ft) of water were needed. At this point, the total load of the pipeline was suspended partly from the floats, partly from the barges. It was then lowered, according to orders, to about a meter (39 in) from the bottom. It was in this position that the diver bolted the edge of the joint

FIG. 2. THE "SEA TRAIN" READY TO START.

FIG. 3. THE "SEA TRAIN" ON THE WAY TO GENEVA.

FIG. 4. GENERAL VIEW OF THE PIPELINE SECTION BEFORE IMMERSION AT GENEVA.

to the section previously laid. In fact, the section had to be kept suspended in this way so that the two flanges of the joint could be brought together.

When the joint was finished, the pipeline was laid directly on the bed of the lake; the floats were detached from it by the diver and were raised to the surface by the barges. The floats were then towed to Préverenges to be used over again for the next section, and the cycle began once more, with the same fear of interruption of operations by the wind. In such labors, the engineer will greatly increase his chances of success if he is backed up by an experienced meteorologist.

FUNCTIONS OF THE SANITARY ENGINEERING DIVISION OF THE STATE DEPARTMENT OF HEALTH

BY WILLIAM A. HEALY* AND FRANCIS J. LARIVIERE†

[*Read June 14, 1956.*]

The Division of Sanitary Engineering is, of course, only one of several divisions in the State Department of Health. Altogether there are some twelve different bureaus or divisions within the department, each concerned with its own specialty in the field of public health. The particular duties assigned to our office are actually few in number but are probably among the most important services performed by a state health agency. Specifically, then, water-supply control, sewage disposal, bathing-place sanitation, camp sanitation and nuisance-abatement problems of an engineering nature are the matters which are of direct interest and concern to us.

As would be expected, a major portion of our time is devoted to the control of public water-supply systems. Incidentally, I might inject here that there are now 123 public water-supply systems in the State, and it has been estimated that these various works serve approximately 75% of the population, and the tendency is for this percentage to increase from year to year as population grows. Of this number of supplies 79 are surface sources, either from lakes and ponds or streams. The number of ground-water systems amounts to 44 and in recent years it is apparent that the tendency is toward development of ground-water systems rather than surface supplies. There are two principal reasons for this trend, one of which is the fact that with recreation on the increase, it is more difficult to find a lake or pond which can be utilized alone with simple treatment such as chlorination. Secondly, ground water is apt to be more uniform in character throughout the year and frequently is more readily available close by the area which is to be served than would be a lake source. In the final analysis, of course, there is always the question of eco-

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nomics to be considered and this, depending on local conditions, will be the deciding factor in selecting ground-water development over a surface supply and, of course, *vice versa*.

Most of our authority, or probably a better word is "responsibility," for supervising public water-supply systems, is derived from Chapter 148, R.S.A., which deals with the protection of sources of water and ice supplies. This statute cloaks the Board with ample authority to investigate the sanitary conditions and methods relating to the source, treatment and distribution of all public water supplies. There are two obvious approaches to the problem. One is by inspections of water-supply facilities in the field, and the second is laboratory analyses of samples collected from the source and from various points in the system. Although, at the present time, there are only five engineers assigned to the Division, we do make an attempt to visit each treatment station at least four times a year. By treatment station we mean a chlorination installation or a filter plant, whichever the case may be. Here, of course, the idea is to see that all units are operating effectively and that proper chlorine residuals are maintained. Present practice is to require a residual which is at least 85% free chlorine, and quite frequently a total residual of 0.3 to 0.4 ppm is sufficient to do this. Occasionally, because of the taste resulting from algae growth, it might be necessary to alter the chlorination schedule. As you are aware, some forms of algae can be readily destroyed, and the essential oils which they produce, causing taste and odor, can be effectively absorbed by increasing the chlorine dose. On the other hand, there are some types which, when treated with higher dosages of chlorine, result in tastes and odors far more objectionable than previously. In either event, it is important that the water-works superintendent observe conditions at the source of supply periodically, in an attempt to anticipate periods of algae bloom, and apply copper sulphate in an effort to control and suppress these growths, rather than resort to chlorination of the water as it enters the system as the method of control.

Just a word now as to the policy of the State Board of Health with regard to protection of surface water supplies. As was mentioned earlier, Chapter 148, R.S.A., gives complete authority to the Board in matters dealing with water used for drinking-water supply. Under

sections 15 and 16 the Board is authorized to make whatever regulations it may deem best to protect a water supply against any dangerous contamination, and these are in effect when a copy is filed with a town clerk and posted in two or more places in the community. Under this provision many series of regulations have been adopted. In every instance there is the basic stipulation that sewage and other wastes shall be disposed of at a minimum safe distance from the shores of the supply involved. Also quite usually water uses, such as bathing, fishing, ice cutting, etc., are restricted to certain areas away from the intake. Thus it is apparent that the State Board's view is that the water supply should be as clean as possible and as free from other conflicting use as may be reasonable under the particular conditions obtaining around the lake. In some cases the Board or the local agency in control has gone so far as to exclude totally all other activities.

Chlorination of all surface-water sources is an essential safeguard. However, we cannot lose sight of the fact that it is actually preventive in nature and designed only to counteract such light or chance contamination as may arise from time to time even on the cleanest watersheds. Chlorination, at least as it is practiced in New Hampshire, is capable only of destroying mild contamination, and we have had experience on two normally good watersheds wherein its capacity was seriously exceeded. At Concord in 1951 we had a situation where chlorination was ineffective in destroying contamination occasioned by recreational use on the watershed, and this was reported on in detail to this association some time ago. Likewise at Claremont in 1952, we encountered another pollution problem resulting from recreational activity which nullified the chlorination treatment facilities. One factor that occurred in both situations which might be mentioned is that coliform organisms, having once gained access to the distribution system, are extremely difficult to eradicate. Somehow or other, the organisms either become encased in the tuberculation products in the mains or in the joints, where they manage to survive severe doses of chlorine. Doses of as much as 100 ppm with adequate detention periods were ineffective. Samples collected immediately after disinfection were apparently free from coliform growths, but those collected a few days later would demonstrate their presence. Thus it is important to bear in mind that for some time sample collections

from the contaminated distribution system may prove to be unsatisfactory.

As to watersheds tributary to a source of public supply, we attempt to inspect these carefully about once in every four years, to determine whether or not hazards may be present. Admittedly this work is carried on altogether too infrequently, but the size of our staff does not permit any improvement in the schedule. On the larger supplies this situation is perhaps not too troublesome, inasmuch as there is likely to be a full-time superintendent, who has an opportunity to maintain a reasonably good check on changing conditions. In the smaller communities, where the superintendent is only part-time and underpaid, difficulties sometimes arise. Frequently the operator can afford little or no time beyond that of checking chlorination operation, and pollution problems can develop. I have in mind particularly watersheds in the northern and central section of the State, where lumbering is carried on from time to time. The Forestry Division of the Forestry and Recreation Department maintains a licensing system for saw mills, and copies of the permits issued are furnished to our office. By this means we are able to determine when lumbering is being carried on within a public water-supply drainage area, and inspections are made of the installations, with particular reference to sewage disposal at the camp site, location of horse hovels, etc. However, occasionally there is a delay between receipt of the copy of the permit to operate and our visit to the area, and almost inevitably the lumber company want to locate its facilities some place along the banks of a stream tributary to the supply. More recently the Division of Occupational Health has established an inspection system for lumber camps, and this, coupled with cooperative efforts by the U.S. Forest Service, which controls many of the watersheds in the northern section of the State, has brought about improvement.

As to the sampling schedule, every attempt is made to comply with the U.S.P.H.S. sampling schedule. This, as you may know, is based on the population served, an increasing number of samples being required for increased population. We have a range whereby the small supplies are sampled at least once every three months, and this increases progressively until we reach the city of Manchester, with the largest system in the State, which is sampled on

the average of once daily. This latter is not strictly in accordance with the Public Health Service schedule, but a reasonably good picture of quality is obtained and it is about the best that our laboratory facilities will permit. The laboratory procedure calls for examining five standard 10-ml portions and, in addition, plantings are made in the 1-ml and 0.1-ml dilutions. All positive tubes are confirmed in brilliant green bile, and the coliform M.P.N. is reported to the local water superintendent or board of water commissioners. Obviously, should positive findings be unusually high or frequent, this serves as a basis for a prompt check in the field, to determine the cause of contamination.

Another phase in our operations which perhaps would be of interest concerns the matter of cross-connections. By law all cross-connections between an approved public system and a secondary supply must be registered with the Department. At the present writing we have about 85 such connections in the State, and these are protected by conventional systems of double check valves of all-bronze construction. These installations are inspected twice yearly, either by engineers from our office or by officials of the local water department, with letters of instruction to plant management whenever defective valves are noted. The tendency of the State Board, in connection with proposals for new cross-connections, is to discourage them whenever possible. Frequently elevated storage with overhead supply with an air-gap from the public system can be arranged, and this we believe to be superior to the double check or swing-type system of protection. However, as can easily be appreciated, new cross-connections do develop, particularly in the wet industries where, because of convenience, the plant superintendent makes piping changes, creating new cross- or interconnections. As these conditions are noted during the course of our inspection, instructions for corrective action are given to plant management.

The staff also carries on a hospital-inspection program which is similarly designed to discover and correct improper connections. To date the best we have been able to maintain in the way of inspection frequency is about once in every four or five years. We realize that this is inadequate, but the situation is somewhat relieved, inasmuch as most new hospital plumbing equipment is so designed as to avoid de-

fective connections. For example, inspection of the new Concord Hospital disclosed only one major criticism of the plumbing installation. This involved submerged connections to the mechanical laundry washers. The matter was called to the attention of hospital administration and corrective measures will follow. At Sacred Heart and Notre Dame Hospitals in Manchester, where new additions have been built, the plumbing installations were excellent.

By law the State Board of Health is required to promulgate plumbing regulations, the same to be minimum throughout the State where public water-supply systems are involved. Municipalities are authorized to develop and impose regulations which are more stringent, and a number of communities, having regular inspectional staffs, have so regulated. We are unable to offer any regular plumbing inspection service and, except in those cities where a plumbing inspector is employed, the duty of inspecting new and existing plumbing installations rests with the local health officer. Obviously, with little or no pay and only a part of his time devoted to the work, the local health officer is usually not active in this field. Further, of course, the fact that he lacks training in such work is also a handicap.

One major function which perhaps deserves mention at this time is the review and approval of plans for new water systems or major alterations to existing ones. Chapter 148 specifically provides that the State Board of Health shall approve, prior to construction, all plans for new water systems or for substantial changes to existing works. The usual procedure followed is for the consulting engineer, engaged by the community, to discuss the general features of the project with us, prior to preparing the preliminary plans. Based on such a conference, the engineer then proceeds to develop one or more plans for the work. These data are usually contained in a preliminary report, a copy of which is furnished to us as well as to the superintendent or water commissioners in the town. After a review in our office, to determine if the plans are in accordance with good engineering practices so far as protection of the public health is concerned, a memorandum containing our recommendations is submitted to the State Board of Health at one of its regular meetings. The Board, depending upon the facts presented, indicates tentative approval, and the engineer is then prepared to proceed with construction plans. The ma-

terial is again submitted for consideration by the State Board of Health, and in due time a final approval is given.

The same policy exists with respect to plans for sewerage and sewage-disposal plants and, of course, as regards these latter, we are seeing more progress as the Water Pollution Commission program goes forward. For example, within the last few years, sewage-treatment plants have been constructed at Laconia, Meredith, Portsmouth Air Force Base, and Laconia State School, and major changes have been made to the disposal works at Hampton, Keene, New London, Wolfeboro and Durham. Other communities are actively investigating their sewage-disposal problems, such as at Dover, Somersworth and Derry. In the matter of sewage-treatment plant operation, our engineers of course visit these plants regularly, advising the operators as to any needed improvement in operation of the plant. Samples are collected frequently to check efficiency of operation, and naturally, since the Water Pollution Commission is active in the field, close cooperation is maintained with that agency.

Just a brief mention of private water supplies and private sewage-disposal systems may be helpful. We, of course, cannot attempt to assume responsibility for all individual water-supply and sewage-disposal installations. We do, however, as time permits, advise property owners, both in the office and in the field, concerning proper design of such facilities. Also a substantial amount of literature is furnished to people who are interested in these subjects. Each year a large number of water samples are submitted from home owners who are interested in obtaining information as to the quality of their water supplies. Last year, for example, I believe we analyzed about 3,000 samples from private or individual water supplies. This service tends to expand, and one large factor in this expansion is the Veteran's Administration and Federal Housing Administration's requirement for loan approval, that the water supply serving the property be tested and approved by the State Board of Health.

Under the provisions of Chapter 125, R.S.A., any camp or place set apart for recreational use by children is considered to be a juvenile camp and as such subject to license by the Board. Thus any camp catering to five or more children, whether on a day-to-day basis, by the week or by the season, falls within the juvenile-camp category.

At the present time there are some 225 camps scattered throughout the State, and they usually operate during the months of July and August. The Board has developed very detailed regulations, governing the sanitation of these camps, and they are inspected at least once during the operating season by a representative from this division. As would be expected, the principal matters of concern to us are water supply, sewage disposal, food sanitation and bathing. Some guidance concerning the camp program itself, where such would bear some relation to public health, is available from the medical and nursing divisions of the department. The inspectional program is of such long standing that there are very few problems cropping up in this area of our operation. Then too, camp management itself is progressive and, being of a high order, cooperates fully with whatever suggestions may be made by the members of the Health Department staff.

In the matter of bathing-place sanitation, the Board again is charged with the responsibility of approving of plans for artificial pools and is responsible for maintaining good sanitation practices at all public bathing places. There are at least 300 well-established bathing areas in the state, catering to the public either on a municipal or private basis. As with the drinking-water supply, it is important that a thorough knowledge be gained of the watershed tributary to the bathing area in question and that any improper sewage-disposal practices are corrected, either by consultation or instruction from us. Standards have been set up as well for dressing rooms, showers, etc., which are a necessary adjunct to any properly operated bathing place. The processing of plans for new pools or major alterations of existing ones proceeds in the same manner as in the case of public water-supply or sewage-disposal systems. We make a real attempt to visit the swimming pools and bathing areas at least once during the season, and if necessary return visits are made. We are not able to devote the amount of time to this work which is perhaps desirable, but the only solution to this situation would be either to strengthen our own staff, or to assign these duties to the local health agency.

PROGRESS REPORT OF COMMITTEE ON COEFFICIENTS FOR FRICTION IN PIPE LINES

December 7, 1956

To the New England Water Works Association:

This Committee last presented a Progress Report on September 24, 1954, in which the conclusions of the 1935 Committee on "Pipe Line Friction Coefficients" were reviewed, and attention was called to the fact that an increase in rate of reduction and carrying capacity of cast-iron mains had been reported, subsequent to water treatment, particularly in connection with the adoption of filtration and treatment in plants using Great Lakes waters.

As a result of these incidents, a research project had been undertaken—at the instigation of the American Water Works Association—to determine the effect of different materials used in filtration and treatment processes upon the carrying capacity of distribution piping. This project was being financed through the National Health Institute by a grant of Federal funds. The work under this grant was being conducted at the University of Illinois under the direction of Dr. A. M. Buswell.

In order to avoid duplication of work, it was recommended in the 1954 Progress Report that the NEWWA Committee confine its work to the reporting to the Association of the results obtained by others working in this field.

The following information is extracted from papers by T. E. Larson, formerly Head of the Chemistry Subdivision, State Water Survey, Urbana, Ill., who has now replaced Dr. Buswell on this research project.

As the carrying capacity, particularly of cast-iron pipe, is associated with pipe "roughness", Mr. Larson, in his report, defines the types of roughness as follows:—

(1) Slime deposit or growth of bacteria (whether or not maganese or iron is present).

(2) Deposition of silt from untreated water (sand, alumina-silicate clays and microorganisms), the effect of which is probably only slightly greater than calculations based upon the reduction of pipe cross-section would indicate.
(3) Incrustation—a crust over the metal surface which may be in the form of
   (a) tuberculation, a formation of nodules constituting localized corrosion;
   (b) afterprecipitation or uniform deposition of insoluble products.

   Both tuberculation and afterprecipitation may exist together or separately.

Mr. Larson's study deals with incrustation and primarily with tuberculation. The present methods of measuring roughness are principally the Williams and Hazen "C" and the Velocity Factor. The Hazen-Williams "C" depicts the carrying capacity but is not a direct indication of roughness. By substituting the same measured data in the Darcy equation for the friction factor (f) and then in the semi-empirical Colebrook-White formula,

$$\frac{1}{\sqrt{f}} = 1.74 - 2 \log\left(\frac{18.7}{R\sqrt{f}}\right) + \frac{K}{r},$$

the K is representative of the effective height of roughness, equivalent to sand grains of diameter K. The K thus can be considered as a "roughness coefficient" which is comparable for similar roughnesses in pipes of different sizes and should serve as a possible unit for comparing the measurement of incrustation.

Another means of estimating carrying capacity and roughness is to develop the Velocity Factor (average velocity divided by velocity at the center). This approximate determination correlates rather closely with the Hazen-Williams "C". However, as pointed out by E. Shaw Cole, Chairman of the AWWA Committee in charge of this project and a member of the NEWWA Committee, there can be a wide variation from the theoretical relationships between average and center velocities and the Hazen-Williams "C", due to the many variables that exist in field testing. The relationship may be dangerously misleading if the data are not carefully obtained and critically examined. Field testing usually cannot include a sufficient number

of measurements of internal waterway cross-section to determine a true average area.

While both the Velocity Factor and the Hazen-Williams "C" are indicative of the effect of the degree of tuberculation, it is questionable that a precise understanding of the type or quantity of roughness can be obtained by these capacity tests, and Larson concludes that the problem requires the development of a field "tool" to "indicate the corrosive potential of water—not to determine roughness, but to determine corrosivity as distinct from sediment or deposition. This tool must be usable at any place in the distribution system under any condition of flow. It must be sensitive to the effect of supersaturation with calcium carbonate as a corrosive inhibitor (not as an incrustant) and to other inhibitors that may be present. It must also be sensitive to the effect of the possible corrosives which may be present, such as dissolved oxygen or low pH."

A method that shows some promise and that may have limited application involves the measurement of potential charges induced by applied current densities of a few microamperes per square decimeter. Results today are not, however, sufficient to warrant drawing definite conclusions. Mr. Larson indicates one of the basic objectives of his study to be the development of such a "tool".

Through laboratory tests, free chlorine has been found to be corrosive to steel at concentrations above 0.4 ppm, at low velocity, at room temperature, in water of 30 ppm NaCl and 120 ppm alkalinity at pH7. Under these conditions, chloramine in concentrations up to 3.6 ppm was not found to be significantly corrosive.

Improvement of tar coatings as a protection for cast iron is desirable.

JOHN C. ADAMS
E. SHAW COLE
F. W. HALEY
WILLIAM D. MONIE
BYRON O. MCCOY
WILLIAM E. STANLEY
(*Signed*) RICHARD H. ELLIS, *Chairman*

REPORT OF COMMITTEE FOR SURVEY OF GROUND-WATER SUPPLIES IN NEW ENGLAND†

SUMMARY OF SYMBOLS UNDER VARIOUS COLUMN HEADINGS IN ACCOMPANYING TABULATIONS

Estimated Population Supplied

W	—	Winter	S	—	Summer

Type of Well

D	—	Dug	Gw	—	Gravel-walled
T	—	Driven	Gd	—	Gravel-developed
A	—	Deep well in rock	S	—	Spring
Gp	—	Gravel-packed			

Well ends in:

S	—	Sand	R	—	Rock
G	—	Gravel	C	—	Clay

Treatment

(These symbols were taken from "Notes Regarding State Directories of Water Treatment Plant Data," U. S. Public Health Service, Cincinnati, O., May, 1941.)

A	—	Aeration	Fp	—	Filtration, pressure filters
Ac	—	Aeration, coke bed	Fz	—	Filtration, zeolite filter
F	—	Filtration	D	—	Disinfection
Fs	—	Filtration, slow sand filters	Dh	—	Disinfection, by hypochlorite
Fr	—	Filtration, rapid sand filters	Dc	—	Disinfection, by chlorine
N	—	Ammoniation	Ng	—	Ammoniation, with NH_3 gas
Nc	—	Ammoniation, by ammonium compound			
I	—	Iron or manganese removal			
K	—	Chemical dosage for corrosive correction or water stabilization			
Kp	—	Chemical dosage for corrosive correction or water stabilization by alkali feed for pH adjustment			
Kc	—	Chemical dosage for corrosive correction or water stabilization by feed of phosphate compounds			
Ko	—	Chemical dosage for corrosive correction or water stabilization by sodium silicate			

Type of Pump

G	—	Gravity	T	—	Turbine
C	—	Centrifugal	A	—	Air-lift
D	—	Displacement	J	—	Jet

Supplies Not Included in 1949 Report are indicated by asterisk (*).

†Continued from this JOURNAL, 70:4, 383, Dec., 1956.

MAINE PUBLIC GROUND WATER SUPPLIES

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
1	Alfred*	Alfred Water Co.	800	.085	1	200′ from spring fed pond	D	28′	8′ x 10′	G	1952
2	Berwick	Berwick Water Dept.	1,500	.168	2	Edge of Town	D	17′	15′ sq.	G	1901
3	Brownville*	Brownville Water Dist.	400	.015	5	Flat between brook and river	T	16-23	2″	S & G	1948
4	Brownville Jct.	Brownville Jct. Water District	1,230	.025	3	Gravel bed of stream	2 S 1 D	14′	10′ sq.	S	1929
5	Brunswick Topsham	Brunswick & Topsham Water Dist.	12,800	1.3	86	85-500 yd Andros. 1-Woods-50′ from stream	82 T	18′-35′	2½″	S	1906
							2 D	35′	40′	S	1906
							1 G	97′	8″	G on R	1954
6	Calais	Calais Water & Power Co.	7,700	0.6	1	Beside stream in St. Stephen, N.B.	D	20′	30′	R	1906
7	Clinton*	Clinton Water Dist.	800	.04 normal .10 when canning pt. in operation	1	Open field 1000′	G	42′	12″	R	1946
8	Danforth	Danforth Water Dist.	700	Not known	5	On hillside 1.5 miles from town	A	150′ to 350′	1-5″ 4-4″	R	1931
9	Farmington Falls*	Farmington Falls Water Co.	170	.0054	1	Edge of village	G	55′	8″	S	1949
10	Fort Fairfield* This well not yet in use	Fort Fairfield Utilities Dist.	2,500	0.4	1	Field—50′ from stream	G	42′	8″	G	1954
11	Houlton	Houlton Water Co.	7,000	1.05	1	Out of pop. area	G	40′	30″	G	1940
12	Kezar Falls	Kezar Falls Water Co.	1,200	0.06	1	Edge of town	T	240′	8″	G	1909
13	Limerick	Limerick Water Co.	800	0.06	7	Springs in pasture	1 D	12′	10′ x 1	C	1925
						Wells, edge of town	1 T 5 S	560′	10″	R	1913

* Denotes new ground-water supplies since time of last report.

MAINE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis				Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese				
1	1,000	0.2	23	6.2	—	—	None	C	180	0.083	
							Fp, Dh				
2	Not known	0.15	17	7.0	19.0	—	Kp	C	350	0.17	
3	120	.05	11	6.3	—	—	None	D	20	0.015	
4	Not known	.07	30	6.0	13.0	—	Dc	D	25	0.021	
5	700	T	44	7.4	—	T	Dc	T	500	0.65	
6	1,000	.05	26	6.2	11.0	—	None	2 D	1,500	0.60	
								1 C			
7	600	.14	214	7.3	7.0	—	Fz	T	300	0.045	
8	Not known	.35	80	7.1	—	—	Dc	D	40	Unknown	
9	6	.05	43	7.9	—	—	None	D	25	.0054	
10	250	.2	221	7.4	—	—	—	—	—	—	This well not yet in use
11	1,000	.02	130	7.6	—	—	Dh	C	700	0.95	
12	200	T	14	7.5	5.0	—	Kp	1 C	160	.07	
								1 D			
13	Not known	.12	39	6.3	2.0	—	Dc	1 C	42	.03	
								1 D			

MAINE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
14	Lisbon Falls	Lisbon Water Works	3,500	0.38	10	Edge of town	8-T	72′	2½″		1922 T
						1 Lisbon	2-G	1-65′	1-10″	G	1925 T
						9 Lisbon Falls		1-70′	1-12″		1952 G
											1954 G
15	Lubec	Lubec Water District	2,900	185,000	1	Edge of town					
						¾ mile from road	D	24′	10′	G	1931
16	Milbridge	Milbridge Water Co.	675	Not known	4	Woods, 1 mile out	1 D	25′	18′	G	1900
							1 T	100′	4″		1916
							2 S				
17	Monson	Monson Spring Water Co.	500	Not known	3	Hillside	T	53′	6″	R	1911
18	New Sharon	New Sharon Water Co.	180	20,000	1	Center of village	A	405′	8″	R	1944
19	Norridgewock*	Norridgewock Water Dist.	500	0.09	1	Windy Hill Road	G	84′	18″	G	1954
20	North Bridgton	No. Bridgton Water Co.	300	Not known	2	Hill above village	S	—	—	--	—
21	East Millinocket	Northern Water Co.	1,600	0.25	1	Gravel Pit—1000′ from river	G	22′	24″	S	1940
22	North Jay*	North Jay Water Dist.	350	Unknown	2	Well, meadow between 2 brooks	G	29′	8″	S	1947
							S				
23	Northport*	Northport Mtn. Spring Water Co.	800 Summer	0.032	3	Edge of settled area	1-A	187′	8″	R	1917
							2-S				
24	Paris	Paris Hill Water Co.	300	Not known	14	Wells in village	2 A	440′	6″	R	1925
						Springs on hill 1 mile outside	12 S	270′			1930
25	Patten	Patten Water Dist.	350	8,900	1	Center of village	A	135′	8″	R	1935
26	Pittsfield	Pittsfield Water Works	3,400	510,000	4	Woods on river bank	D	3-25′	18′ x 8′	G	1894
								1-20′	53′		

MAINE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
14	G—200 G—365	0 .1	77 73	7.6 7.6	—	—	None	T T	200 300	0.38	These figures are for 2 gravel wells. Driven well analyses are similar. Displacement pump used for driven wells. Daily pumpage is total for all.
15	Not known	0.2	156	7.7	2	—	Dh Nc	2 C	200 300	0.18	
16	Not known	T	19	6.2	16	—	None	D	—	0.015	
17	Not known	.16	23	6.6	5	—	None	3 D	40-50	0.015	
18	40	.08	44	8.0	—	—	None	D	32	Unknown	Automatic. No record of number of hours pumping.
19	245	T	54	8.3	—	Neg.	None	T	350	0.09	
20	Not known	.05	30	6.7	4	—	None	None	—	—	Gravity supply
21	800	T	40	7.3	—	—	—	C	350	0.25	
22	250	.2	21	6.4	—	—	Dh	T	150	Unknown	Supply partly gravity
23	30	6.0	93	7.0	—	—	Kc	D	28	0.03	
24	Not known	0.25	19	6.5	5	—	None	D	15	Unknown	Supply partly gravity
25	Not known	T	201	7.4	11	—	None	D	13	8,900	
26	Not known	.05	31	6.5	11	—	Dc	1 D 1 C	300	510,000	

MAINE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
27	Harrington	Quantabacook Water Co.	600	Not known	1	Woods—1.5 mi. from town	S	8′	15′ x 25′	Sand	1911
28	Richmond*	Richmond Water Works	1,500	0.103	1	Dresden, near Kennebec River	G	61′	12″	On ledge	1954
29	Sanford	Sanford Water Dist.	17,000	1.0	33	Woods—1.5 mi. from town	T	10′-30′	1-8″ 4-6″ 5-3″ 23-2½″	 G 	1902 1926 1934 1937
30	South Berwick	So. Berwick Water Co. Inc.	2,500	0.145	2	Woods—1 mi. from town	G	53′	12″	R	1942
31	South Freeport*	So. Freeport Water Dist.	600W 650S	0.027	1	Woods—1 mi. from town	S	18′	10′ x 10′	G	1946
32	South Paris	So. Paris Water Dept.	3,000	1.0	3	Glacial drift Edge of town	G	47′	24″	G	1936 1937 1945
33	Port Clyde	St. George Water Co.	75W 300S	0.10	2	Edge of village	A	200′ 150′	6″	R	1911 1952
34	Waldoboro	Waldoboro Water Co.	1,100	0.025	4	Open—2 mi. from town	A	3-100′ 1-200′	8″	R	1900
35	Warren	Warren Water Co.	800	0.035	2	Woods—1.5 mi. from town	A	1-200′ 1-123′	6″	R	Not known
36	Washburn	Washburn Water Co.	1,000	0.04	3	Field—100′ from stream	T	34′-38′	6″	G	1916
37	West Paris*	W. Paris Water Dept.	400	0.03	1	Edge of town	G	42′	8″	R	1951
38	Yarmouth*	Yarmouth Water Dist.	2,600W 2,800S	0.25	1	Open country 1 mile from highway	G	98′	12″	G	1947

MAINE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
27	Not known	.27	38	6.6	7	—	None	None	—	—	Gravity supply
28	650	T	34	6.5	—	Neg.	None	C	200	.105	
29	50-100	.05	25	6.0	—	.017	Fz-Dc	C	1-350	1.0	
							Ng		1-700		
30	120	.4	31	6.5	21	—	Dh	T	2-200	0.145	
31	200	.11	32	6.8	—	—	None	T	36	0.027	
32	650+	.14	35	6.3	—	—	None	T	3-500	1.0	
33	Not known	.65	50	6.8	6.0	—	None	D	100	0.1	
34	Not known	.15	26	6.6	5.0	—	Dh	D	45	.02	
35	Not known	.05	162	7	6.0	—	None	D	45	.035	
36	675	0	208	7.9	4.0	—	None	D	500	.04	
37	175	3.4	36	6.3	—	1.3	Dh				
							Kc	T	150	.03	
38	1,000	T	38	7.2	2	—	None	T	300	.25	

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
1	Alton	Alton Water Works	1,200	.072	1	In valley of river	G	40′	48″	G	1938
2	Alton	Alton Water Works			1	On Hill	A	300′	6″	R	1927
3	Amherst*	Amherst Village Dist.	1,400	.040	1	Near brook	Gp	225′	8″	G	1951
4	Bath	Bath Village Water Co.	145	.009	1	Marsh, Silty Soil	D	10′	10′	S	1878
5	Belmont	Belmont Water Works	600	.082	1	Lowland nr. Valley	G	60′	48″	G	1938
6	Bennington	Bennington Water Works	500	.038	3	On Hill	D	7′	50′	S	
7	Berlin*	Berlin Water Works	17,000	2.7	1	Near river	Gp	49.6′	24″	G	1946
8	Bristol*	Bristol Water Works	1,200	.10	1	E. of town	Gw	49′	12″	G	1950
9	Campton	Beebe River Supply	550	.072	1	Foot of Hill	D	9′	55 x 19′	G	1926
10	Colebrook	Colebrook Water Co.	1,500	.099	10	River bank	T	52′	2½″	S&G	1926
11	Concord	Concord Water Works	25,000	3.0	150	River Valley	T	38.5′	2½″	S	1930-31
12	Conway	N. Conway Precinct	2,000	.120	1	Brook Valley	D	18′	10 x 10′	S	1928
13	Conway*	Conway Village Fire Pre.	1,500	.13	1	Near Air Field	Gp	66′	18″	S&G	1955
14	Derry	Derry Water Works	4,500	.225	10	Brook Valley	D	16′	1-25′ 9-45′	S	1909-40
15	Dover	Dover Water Works	16,000	1.2	1	Sandy Plain	G	82′	60″	G	1940
					1	Near Lake	G	65′	48″	G	1947
16	Dover*	Dover Water Works			1	Near Willand Pond	Single Cased T	56′	8″	G	1955
17	Epping	Epping Water Works	1,200	.079	16	River Valley	T	47-54′	2½″	G	1927

* Indicates either new or additional sources of Ground Water Supply since the 1949 report.

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
1	700	0.05	44	6.5	30		—	T	200	—	Auxiliary supply
2	30		42					No pump	No pump	—	Free flowing well
3	—	0.0	28	7.1	12	—	None	T	200	0.04	
4		0.05	47	6.6	15		—	—	G	—	
5	500	0.05	20	6.3	25		—	T	660	.081	
6		0.05	18	6.8			—	—	G	—	Augmented by surface supply
7	1,000	0.1	58	6.1	—	—	Cl_2	T	330 & 1,000 (2 stage)	13 days 24 hours	Auxiliary supply
8	350	0.1	12	6.3	—	—	None	T	350	0.10	
9	500	0.05	18	6.8	6		—	D	60	.072	
10		0.05	50	7.2	6		D	D	250	.098	Augments surface supply
11	1,500	0.10	10	6.4	8		K	C	1,000	—	Auxiliary supply
12			17	6.8	7		—	C	100	.108	Auxiliary supply
13	800	0.1	14.0	6.7	36	0.0	None	—	—	—	Presently under construction
14		0.15	60	6.3	37		D	C	550	.225	
15	970	1.0	37	6.6	13		D	T	870	.750	
	350	.00	20	6.2	20		—	T	350	.400	
16	350	0.1	20	6.3	5	—	Dc,F	T	350	—	Presently under construction
17		0.4	78	7.1	11		—	D	147	.079	

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
18	Exeter*	Exeter Water Works	6,300	0.4	1	Near River	Gp	60′	24″	R&G	1951
19	Farmington	Farmington Water Works	2,500	.097	1	Brook Valley	D	30′	20′	R	1938
					1	Brook Valley	G	55′	48″	G	1943
20	Francestown	Francestown Electric & Water Co.	150	.009	2	Hillside	D	12′	15′	S	1922
21	Franconia	Franconia Water Works	450	.02	1	Side of Hill	D	10′	50 x 100′	S	1937
22	Franklin	Franklin Water Works	6,000	.24	17	Foot of Hill	D	6′	5′	G	1938
23	Gorham	Gorham Alpine Aqueduct Co.	300	.018	6		D	10 x 15′	10′	S	1873
24	Gorham*	Gorham Water Works	3,000		1	Near River	Gp	50′	12″	G	1949
25	Greenville	Greenville Water Works	1,250	.044	1	River Valley	D	18′	20′	C	1915
26	Hampton*	Hampton Water Works Co.	3,500 W 21,500 S	.450	9	Coastal Plain	D Gp	23′	10′	G	1908-51
27	Haverhill	Haverhill Precinct	400	.024	6	Sandy Plain	D	10′	8′	S	
28	Haverhill	N. Haverhill Water Works	200	.03	7		D	10′	8′	C	1938
29	Henniker	Henniker Water Wks.	1,200	.072	1	Foot of Hill	D	25′	25′	G	1914
					1	Foot of Hill	T	40′	6″	S	1947
30	Hill	Hill Water Works	400	.035	1	River Valley	G	45′	24″	G	1940
31	Hopkinton*	Hopkinton Village Aqueduct Ass'n	200		10	Near Brook	7D 3T				
32	Hudson	Hudson Water Co.	2,500	.15	28	Brook Valley	26T & 2D	29′	D2½″	S&G	1895 1923, '35
33	Keene*	Keene Water Works	15,600	2.0	1	W. Keene	Gp	76′	12″	G	1908, '53
34	Lyme	Lyme Aqueduct Co.	200	.012	1		D	8′	18′ x 20′	S	

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
18	600	0.2	72.0	7.6	—	—		T	460	0.33	
19		0.05	20	6.3	25		—	C	270	.05	
	300	0.05	20	6.2	25		—	T	200	.047	
20		0.05	25	6.6					G		
21		0.05	20	6.9			—		G		
22		0.05	42	6.3			—		G	.125	This source supplies 50% of use.
23			14	6.8	4		—		G		
24	300	< 0.1	22.0	7.0	—	—	None	T	500	10 weeks 10-12 hr./day	Auxiliary supply
25	40	0.02	20	6.6	19		D		65	.043	
26		< 0.1	48	6.4	—	—	None	D & T		.45	
27		0.00	45	7.4	4		—		G		
28	40	0.05	32	7.1			—		G	.03	
29		0.05	18	6.8			—	C	165	.036	
		.00	20	6.7			—	C	150	.036	
30		0.15	27	6.7	7		—		150		
31		< 0.1	20	6.7	—	—	None	—	—		Free flowing wells
32		0.10	34	6.4	22		—	D	180		
33	700	< 0.1	36	6.1	—	—	None	C	700	1.0	
34		0.05	35	6.8	13		—		G		

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
35	Marlboro	Marlboro Water Wks.	350	.017	7	Hillside	D	8′	10 x 8′	S	1908-28
36	Marlboro*	Marlboro Water Wks.	600	.75	1	Near River	Gw	46′	12″	G	1953
37	Merrimack	Merrimack Company	245	.012	10	River Valley	T	18′	2½″	S	
38	Merrimack	Shedd-Harris Mem. System	400	.02	1	Foot of Hill	D	10′	20 x 20′	S	1934
39	Milford	Milford Water Wks.			1	Brook Valley	G	45′	42″	R	1934
40	Milford	Milford Water Wks.	3,600	.35	3	Brook Valley	D	12′	1-18′ 2-30′	R	1889-93
41	Milton	Milton Fire District	600	.03	12	Near Lake	T	25′	2½″	G	1936
42	Nashua	Pennichuck Water Works	30,000	2.63	21	Brook Valley	D	16-52′	19-2½″ 1-6″ 1-1½″	G	1898
43	Newfields*	Newfields Village Pre.	< 10		1	Near Brook	Gp	24′	8″	G	1954
44	Pembroke*	Pembroke Water Wks.	3,000		1	Near River	Gp	76′	24″	G	1950
					1	Near River	Gp	70′	24″	G	1949
45	Peterboro*	Peterboro Water Wks.	2,500	.25	1	Near River	Gw	69′	24″	G	1953
46	Plymouth*	Plymouth Village Fire Precinct	2,200	.25	1	Near River	Gp	44′	24″	S&G	1951
47	Portsmouth	Portsmouth Water Works	26,000	2.2	50	Sandy Moraine	T	30-50′	2½″	G	1892
48	Portsmouth	Portsmouth Water Works			4	Sandy Moraine	T	60′	8″	G	1889
49	Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	70′	36″	G	1922
50	Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	70′	36″	G	1922
51	Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	54′	36″	G	1934

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
35		0.10	19	6.4	20		—			Gravity	
36	125	0.1	34	6.7	—	—	None	T	100	.075	
37		1.20	25	6.6	18		A.F.	D	70	.012	
38		0.05	50	7.3	3		—	C	150	.02	
39	200	0.05	24	6.8	7		A.F.		550	.35	
40	250	0.25		6.4	15						
41		0.05	20	6.8	10		K	C	250	.03	
42	4,500	0.10	31	6.9	2		D				Free flowing wells & springs
43	50	< 0.1	24	6.6	13.9	—	None		50		Primarily for fire protection
44	800	0.0	19	6.6	8.0	—		C	700	1.0	
	800	2.4	18	6.1	—	—			700	—	Auxiliary source
45	400	< 0.1	10	6.4	25.3	—	None	T	300	Varies	Also utilize a surface water supply
46	—	0.1	30	6.6	—	—	None	T	350	.25	
47	200-350	0.70	73	7.4	6		D	D	1,740	2.5	
48		0.05	59	7.6	3		D	D	700		
49	500	0.05	73	7.7	3		D	T	420		
50	450	0.05	81	7.7	3		D	T	125		
51	220	0.60	47	7.4	4		D	T	240		

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
52	Portsmouth	Portsmouth Water Works			55	Sandy Moraine	T	35-50′	2½″	G	1941
53	Portsmouth*	Portsmouth Water Dept.			4	Coastal Plain Near Brook	Gp	40-90′	24″	G	1955
54	Raymond	Raymond Water Wks.	725	.044	1	River Valley	D	30′	25′	S	1930
55	Rollinsford	Salmon Falls Water Co.	800	.03	2	Brook Valley	D	202-213′	8″	R	1920
56	Somersworth	Somersworth Water Co.	6,200	.35	2	River Valley	G	70′	60″	R	1935-39
57	Tamworth	S. O. Kimball Supply	100	.005	1	River Valley	D	7′	20 x 12′	G	1937
58	Wakefield	Sanbornville Water Works	550	.025	1	Brook Valley	D	18′	20′	S	1939
59	Walpole	Walpole Water & Sewer Co.	500	.025	6	Brook Valley	T	20′	2½′	S	1940
60	Walpole	N. Walpole Precinct	1,200	.060	3	Brook Valley	D	8′	8′	S	
61	Warren*	South Main St. Water District	80	.01	1	Near River	D	16′	36″	S&G	1954
62	Whitefield	Whitefield Village Fire District	1,200		4	Hillside	4A & 3D	(D8-12′)	10″	R	1935
63	Winchester*	Winchester Water Dept.	1,500	.321	1	Near Brook	Gp	56′	24″	G	1949

NEW HAMPSHIRE PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
52		0.10	74	7.4	5		D	C	500		
53	2,500	< 0.1	84	7.3	—	—	Dc	T	2,500 (Total)	—	Not yet in operation
54		0.05	37	6.3	23		—	C	250	.043	
55	40	0.05	47	7.0	3		—	D	35		
56	600	0.10	31	7.0	3		K	T	600		
57		0.05	18	6.4	20		—		G		
58		0.05	46	6.8	12		—	T	150	.025	
59		0.05	35	7.0	8		—	D	60		Auxiliary supply
60		0.05	43	6.7	8		—	D	150		
61	60	3.0	30	6.2	—	—	Dh	C	50	0.01	
62	325	0.00	26	7.2	3		—	T			
63	375	< 0.1	28	6.6	—	—	None	C	350	0.321	

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
1	Barrington	Bristol County Water Co.	8,000	0.693	1**	Isolated	Gp	83′	12″	G	1950
					1**	Isolated	Gp	81′	12″	G	1951
2	Burrillville Glendale New Village	Harrisville Combing Co.	105	0.025	1	Isolated	A	419′	8″	R	1937
3	Burrillville (Harrisville)	Harrisville Fire District	1,050	0.089	1	Near brook	Gd	49′	14″	G	1948
4	Burrillville (Mapleville)	Stillwater Worsted Mills	250	0.018	1	Not isolated	A	200′	6″	R	1926
5	Burrillville (Mohegan)	Wanskuck Co.	200	0.017	1	Near river	A	250′	6″	R	1905
					1	Near river	A	310′	6″	R	1905
6	Burrillville (Oakland)	Oakland Worsted Co.	200	0.021	1	Near river	A	230′	8″	R	1911
					1	Not isolated	A	160′	6″	R	1916
7	Burrillville (Oakland-Robinson)	Mrs. Earle B. Robinson	120	0.005	1	Not isolated	D	22′	30″	G	1910
8	Burrillville Pascoag	Pascoag Fire District	2,400	0.152	1	Near river	Gp	46′	12″	G	1946
					1**	Near river	Gp	45′	12″	G	1948
9	Cumberland	Town of Cumberland	7,000	0.394	1**	Near river	Gp	84′	18″	G	1950
10	East Greenwich	Kent County Water Authority	12,000	1.246	1	Near river	Gp	107′	12″	G	1941
					1	Isolated	Gp	118′	12″	G	1943
11	East Providence	Town of East Prov.	35,000	3.944	1**	Near river	Gp	60′	16″	G	1950
					1**	Near river	Gp	73′	16″	G	1951
					1**	Near river	Gp	58′	16″	G	1951
12	Exeter (R.I. Training School)	State of R.I.	1,000	0.162	1	Isolated	Gp	46′	12″	G	1947
					1**	Isolated	Gp	50′	12″	G	1948

** Developed since last report.
Note: Does not include supplies serving fewer than 100 persons.

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
1	700	0.7	31	5.7		0.0	I,AC	T	700	0.542	
	700	0.6	25	5.8		0.1	Fp,Dc	T	700	0.151	
2	30	0.1	36	6.8		0.0	—	D	30	0.025	
3	160	0.2	14	5.7		0.0	Dh	T	100	0.089	
4	65	0.0	39	7.0		0.0	Dh	T	50	0.018	
5	3	1.1	40	6.8		0.0	—	D	3	Not used	
	20	0.1	31	7.3		0.0	—	T	20	0.017	
6	12	0.1	32	5.8		0.0	—	T	10	0.008	
	14	0.0	28	5.6		0.0	—	T	12	0.013	
7	15	0.0	50	5.6		0.0	—	C	9	0.005	
8	285	0.2	36	5.6		0.0	Dh,Kc	T	250	0.152	
	160	0.1	30	5.7		0.0	Dh,Kc	T	150		
9	1,100	0.2	88	6.4		2.4	Dh	T	750	Not used	Surface water main source of supply
10	1,320	0.0	22	5.7		0.0	Dc,Kp	T	1,500	0.541	
	2,700	0.0	18	5.7		0.0	Dc,Kp	T	1,500	0.705	
11	750	0.0	40	6.1		0.0	—	T	600		
	760	0.0	50	6.0		0.0	—	T	580	1.850	Also surface water supply
	1,280	0.0	40	5.7		0.0	—	T	1,500		
12	—	0.0	34	5.6		0.0	—	T	150	0.066	
	—	0.0	15	5.6		0.0	—	T	200	0.096	

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
13	New Shoreham	Block Island Water Co.	200 W	0.070	1**	Near pond	Gp	135′	6″	S&G	1950
			2,000 S		1**	Near pond	Gp	107′	6″	S&G	1950
14	N. Kingstown	Town of N. Kingstown	6,000 W	0.432	1	Isolated	Gp	50′	12″	G	1942
			7,500 S								
15	N. Smithfield	Forestdale Pump Co.	250	0.008	1	Isolated	A	750′	10″	R	1915
16	Pawtucket	City of Pawtucket	125,000	13.882	1**	Not isolated	Gp	107′	10″	G	1949
					1**	Near river	Gp	132′	12″	G	1949
					1**	Near river	Gp	50′	16″	G	1950
					1**	Near river	Gp	80′	12″	G	1950
17	Richmond (Alton)	Richmond Lace Works	190	0.040	1	Not isolated	D	12′	120″	G	1916
18	Smithfield	Northwestern Water Co.	550	0.027	1	Isolated	Gp	50′	6″	G	1946
					1**	Isolated	Gp	47′	6″	G	1948
19	S. Kingstown	Wakefield Water Co.	7,300	0.759	1	Isolated	Gp	75′	12″	G	1946
					1	Isolated	Gp	55′	12″	G	1946
					1**	Isolated	Gp	54′	12″	G	1948
20	S. Kingstown (Kingston)	Bernon Lemoine Water	600	0.026	1	Isolated	S	6′	5″	S&G	1930
21	S. Kingstown (R.I. Univ.)	State of R.I.	1,500	0.147	1**	Near pond	Gp	133′	12″	G	1948
22	Tiverton	North Tiverton Fire District	4,400	0.129	30	Swamp	T	18′	2½″	S&G	1928
					10	Swamp	T	25′	2½″	S&G	1937
					10**	Swamp	T	37′	2½″	S&G	1950
23	Warwick & Coventry	Kent County Water Authority	23,500	1.667	1	Near river	Gp	38′	12″	G	1941
					1	Near river	Gp	58′	18″	G	1942
					21	Near river	T	32′	2½″	G	1945

** Developed since last report.

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
13	42	3.8	43	6.1		0.2	Dh	T	42	—	Wells used only to meet peak demand
	38	0.2	32	6.0		0.0	Dh	T	38	—	
14	1,400	0.0	19	5.7		0.0	Dh,Kp	T	1,000	0.432	
15	10	0.3	65	6.5		0.0	Dh	D	10	0.008	
16	700	0.0	54	5.8		0.0	—	T	800	0.261	
	1,400	0.1	56	6.4		0.1	—	T	1,400	1.501	Surface water main source of supply
	1,400	0.0	42	6.1		0.0	—	T	1,400	Not used	
	700	0.0	44	6.1		0.0	—	T	700	Not used	
17	125	0.0	30	5.3		0.0	Dh	C	100	0.040	
18	—	0.1	14	5.9		0.0	Dh	T	120	0.027	
	—	0.1	14	5.8		0.0	Dh	T	120		
19	1,480	0.0	36	5.7		0.0	Dh,Kc	T	500	0.759	
		0.0	16	5.6		0.0	Dh,Kc	T	800		
		0.1	18	5.7		0.0	Dh,Kc	T	500		
20	—	0.1	16	5.6		0.0	—	D	50	0.026	
21	950	0.1	35	6.2		0.0	—	T	500	0.147	
22	175	0.3	25	5.7		0.0	Dh	C	215	0.129	
	96	0.1	46	5.8		0.0	Dh	C	96		
	115	0.0	59	5.9		0.0	Dh	C	88		
23	555	0.1	25	5.6		0.0	Dh,Kc	T	500	0.895	Surface water also utilized
	275	0.7	15	5.7		0.0	Dh,Kc	T	250		
	640	0.1	25	5.6		0.0	Dh,Kc	T	415		

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
24	Westerly	Town of Westerly	17,500	1.673	65	Near river	T	45′	2½″	S&G	1897
					87	Near river	T	60′	2½″	S&G	1943
					1**	Near river	Gp	60′	24″	S&G	1954
25	Westerly (Bradford)	Bradford Dyeing Association	400	0.144	7	Isolated	T	65′	2½″	S	1913
26	Westerly (Bradford)	Town of Westerly	450	0.062	25	Near river	T	55′	2½″	S	1937

** Developed since last report.

RHODE ISLAND PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
24	—	0.2	22	5.9		0.0	—	C	1,200	1.262	
	—	0.0	25	5.8		0.0	—	C	1,200	0.411	
	750	0.3	45	6.8		0.0	—				
25	—	0.0	41	6.1		0.0	—	D	147	0.144	
26	375	0.0	44	5.7		0.0	—	C	250	0.062	

VERMONT PUBLIC GROUND WATER SUPPLIES

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
	Bennington Co.										
1	Bennington	Village	10,600	1.5	1	E. of vill. near upper reservoir	A†	52′	8″	G	1954
2	No. Bennington	Village	2,180	0.2	1	E. of S. Shaftsbury res., in woods	D	14	96″	G	1940
	Caledonia Co.										
3	Hardwick	Village	1,695	0.5	1	W. side of vill. in sandpit	G	48	24″	S	1940
4	Lyndonville	Village	1,500	0.4	1	N. end vill., E. side of R.R.	G	54	24″	S	1937
5	St. Johnsbury	Village	7,367	1.4	1	N. end of vill., former gravel pit	G	68		G	1949
	Chittenden Co.										
6	Fort Ethan Allen	U. S. Govt.	2,000	0.3	4	Flat plain, W. of Winooski River	3A	400	8″	3R	—
							1G	30		1S	1941
7	Shelburne	Fire District	600	0.15	1	Ctr. of vill. at school	A	330	8″	R	1940
8	So. Burlington	Private water co.	200	0.03	1	Above Lake Champlain at Queen City	A	280	8″	R	1946
9	Winooski	City	6,725	0.4	9	Around res. N.E. of city	3A	450		R	1905
							6T	15		S	1949
	Essex Co.										
10	Beecher Falls	Private water co.	200	0.05	1	E. of vill. beside Conn. R.	G	65	18″	S	1948
	Franklin Co.										
11	Enosburg Falls	Village	1,290	0.3	1	2.4 mi. N.E. of vill.	G	35	24″	S&G	1949
	Lamoille Co.										
12	Morrisville	Village	2,000	0.4	2	N. of vill., S. side Lamoille R.	G	30	24″	S	1942
							G	30	24″	S	1952

† Drilled well in gravel.

VERMONT PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
1	200							T	280	0	Auxiliary well has not yet been used
2	—		13	6.0	—	—	Kc	C	150	0.1	Part-time operation
3	500	.06	122	7.9	—	—	—	T	500	0.3	Auxiliary, can supply 100% of need
4	—		130	7.6	—	—	—	T	400	0.2	Auxiliary, can supply 100% of need
5	1,236		250	7.6	—	—	—	C	800		Auxiliary supply, seldom needed
6	—	—	108	7.6			Dh	T	500	0.3	Normal supply
		.2	42	7.5			Dh	T			Auxiliary supply
7		.28	140	7.6	—	—	—	T	60	0	Emergency supply
8		—	167	7.7	—	—	—	T	100	0.03	100% of community supply
9		—	106	7.5	—	—	Dc	T		0.15	Auxiliary with surface water supply
		—	69	7.6	—	—	Dc	C	200		
10	—	—	38	7.4	—	—	—	T	230	.05	Total supply
11	800	—	123	7.8	—	—	—	T	500	0.3	Total supply. Surface-fed reservoir serves as auxiliary
12	750	—	70	7.0	—	—	—	T	500	0.4	Total supply
									500		Auxiliary

VERMONT PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
	Orange Co.										
13	Bradford	Village	695	0.2	1	S. of Waits R.	G	25	24″	G	1949
14	Chelsea	Private water co.	30	0.01	1	S. of vill. W. of White R.	T	10	1½″	S	1954
15	Randolph	Village	2,220	0.4	1	Ctr. of vill. S. of R.R.	A	196′	8″	R	1941
16	Wells River	Village	600	0.1	2	1—Ctr. of vill. S. of R.R. & 1—N. of village	A D	84		R	1897
	Orleans Co.										
17	Newport	City	5,200	0.8	2	1—N.E. part of city E. side South Bay & 1—Ctr. of city, N.E. City Hall	G G	47 72	18″ 24″	G G	1949 1954
18	Newport Center	Village	250	0.05	6	S. side village	D	18	2″	S	1935
	Rutland Co.										
19	Brandon	Village	2,800	0.5	1	Forestdale	G			G	1952
20	Castleton	Village	430	0.1	1	N. side vill., No. of Castleton R.	G	31	24″	G	1939
21	W. Rutland	Village	2,300	0.5	1	S.E. side of vill.	T	25	12″	G	1950
	Washington Co.										
22	Cabot	Village	200	0.03	1	E. of Walden Hghts. Rd., N. of village	A	232	6″	—	1949
23	Graniteville	Village	980	0.15	1	S.E. of village	A	—	—	—	1950
24	Northfield	Village	2,250	0.35	2	S. of vill., E. side Dog R.	G G	30 30	24″ 24″	G G	1939 1948
25	Waterbury	Village	3,300	0.5	2	1—Ctr. of vill. in park & 1—S. end vill. at canning factory	G A	32 530	24″ 8″	G R	1938 —

VERMONT PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
13	800	0.0	86	7.4	—	—	—	T	500	0.1	Auxiliary supply, part time
14	—	—	—	—	—	—	—	D	—	0.01	Auxiliary supply, part time
15		—	113	7.4	—	—	—	T	400	0.2	Auxiliary supply, part time
16		0.1	145	7.6	—	—	—	C		0.1	Total supply
17	—	—	75	7.7	—	—	—	T	300	—	Auxiliary supply
	600	0.02	143	8.0		0	—	T	550	0.5	Principal supply
18		—	226	7.6	—	—		—	—	0.05	Total supply
19											Auxiliary supply, part time
20	—	—	84	7.0	—	—	—	T	400	0.1	Total supply
21	—	0	134	7.5	—	—	—	C	100	.14	Auxiliary supply, part time
22	—	—	132	7.9	—	—	—	T	43	.03	Principal supply
23	—	—	—	—	—	—	—	—	—	0.1	Auxiliary supply, part time
24	—	0.02	45	6.8	—	—	Dh	T	500		1 well auxiliary
								T	500	0.35	1 well principal source
25	—	0.1	105	6.5	—	—	Dc	T	500	0.00	Auxiliary, seldom used
		0.3	164	7.4	—	—	—	T	360	0.18	Auxiliary, part time

VERMONT PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	City or Town	Utility	Population Served	Present Consumption M.G.D.	No. of wells	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction
	Windsor Co.										
26	Bethel	Village	1,200	0.2	1	½ mi. N. of vill., W. of White R.	G	69	12″	S	1949
27	Bridgewater	Private water co.	120	0.02	1	Rear of mill in vill.	A	200	8″	R	
28	Cavendish	Private water co.	150	0.02	1	Rear of mill in Cavendish	A	240	8″	R	
29	Chester	Village	1,000	0.15	1	In vill., N. of Williams R.	G	47	18″	S	1950
30	Proctorsville	Village	600	0.1	1	W. of vill. near Black River	G	30	12″	S	1947
31	Quechee	Private water co.	210	0.03	1	Near mill in vill.	A	401	8″	R	
32	Rochester	Village	500	0.14	1	E. of White R., S.W. of vill.	G	40	12″	G	1950
33	Springfield	Village	5,500	0.8	30	N. of vill., W. of Black R.	T	25	2″	S	1941
					2	⅛ mi. N. of shallow wells	G	30	18″	S	1954
34	White River Jct.	Village	3,400	0.5	2	1—1 mi. W. of vill. & 1—in sand pit in Wilder Village	G G	35	12″	S G	1949 1955
35	Windsor	Village	3,500	0.5	2	N.W. side of vill., N. of pumping sta.	G	25		G	1947 & 1948

VERMONT PUBLIC GROUND WATER SUPPLIES (*Continued*)

No.	Test Yield g.p.m.	Analysis					Treatment	Type of pump	Capacity of pump g.p.m.	Present daily pumpage, M.G.	Remarks
		Iron	Hardness	pH	CO_2	Manganese					
26	150	0.0	72	7.2	—	—	—	—	200	0.1	Principal source
27		0.0	100	7.7	—	—	—			0.02	Only source of supply
28		0.0	82	7.8	—	—	—			0.02	Principal supply
29			22	7.0				T	300	0	Auxiliary supply, part time
30			66	6.6				T	100	0.1	Total supply
31	—	0.02	154	7.4	—	—	—	T	8	0.03	Total supply
32	—	0.0	30	6.6	—	—	—	T	100	0.1	Auxiliary supply
33	—	—	60	7.1	—	—	—	C	750	0.7	90% of summer supply Auxiliary at times
34	—	—	97	7.5	—	—	—	T T	200	0.2	Auxiliary
35			130	7.6	—	—	—	T	400 & 400	0.4	1 well used majority of time as partial supply, 1 well auxiliary

REPORT OF COMMITTEE ON RAINFALL AND YIELD OF DRAINAGE AREAS

RAINFALL IN NEW ENGLAND*

PART VI. RAINFALL IN CONNECTICUT

ANSONIA, CONN. (Ansonia Water Co.)—Elevation 40 feet
Record: January, 1923, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.31	4.37	8.09	0.89	3.44	2.73	5.89	8.24	3.07	3.54	4.93	5.91	54.41
1943	3.35	2.01	5.19	3.06	4.98	4.09	2.94	1.58	2.23	6.40	4.56	1.46	41.85
1944	2.85	2.11	7.63	4.22	1.74	2.45	0.32	1.60	11.39	1.85	6.60	4.64	47.40
1945	—	—	—	—	—	—	7.23	3.30	3.53	—	6.96	5.87	—
1946	2.58	2.98	1.65	2.36	7.48	2.98	3.67	5.84	3.61	1.35	1.47	2.51	38.48
1947	2.72	2.37	3.75	—	4.48	4.55	3.23	4.20	2.96	3.54	7.82	3.85	—
1948	4.27	1.73	5.27	7.75	—	4.45	3.22	2.75	0.27	1.62	6.30	6.59	—
1949	7.47	2.69	3.43	4.43	5.85	0.20	3.20	2.52	4.37	1.55	1.73	2.65	40.09
1950	3.23	3.74	3.01	2.87	4.57	4.76	4.78	5.86	0.99	1.86	5.32	4.85	45.84
Av. 25 yrs.	4.09	3.38	4.17	4.04	3.75	3.89	3.47	4.44	4.20	3.29	4.08	3.71	46.51

BAKERSVILLE, CONN. (New Hartford)—Elevation 710 feet
Record: May, 1912, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.96	2.82	7.20	0.65	5.66	4.80	6.03	2.74	4.09	3.66	6.81	5.39	51.81
1943	3.20	1.83	3.36	2.64	6.04	2.88	2.62	1.83	2.27	5.35	4.68	0.81	37.51
1944	1.79	2.02	4.59	4.36	1.68	4.03	2.51	2.66	6.64	2.15	6.30	4.03	42.76
1945	3.53	4.06	2.07	5.36	7.54	5.78	6.22	4.93	4.13	2.43	5.41	4.32	55.78
1946	2.99	2.25	1.69	2.22	5.80	2.53	6.76	3.36	3.72	1.57	0.94	2.45	36.28
1947	2.89	2.30	3.19	3.96	4.90	3.46	6.07	1.59	2.91	2.46	7.22	2.55	43.50
1948	3.24	2.25	3.58	3.77	6.89	4.45	3.22	2.75	0.27	1.62	6.30	6.59	44.93
1949	7.88	2.76	2.96	3.88	6.91	1.44	2.41	3.04	3.02	2.39	1.68	3.39	41.76
1950	4.42	4.59	3.55	2.65	3.63	6.10	3.48	4.72	1.99	1.95	7.69	5.22	49.99
Av. 38 yrs.	3.65	2.91	3.59	3.67	4.19	4.11	4.14	4.13	4.13	3.31	4.07	3.38	45.28

BALTIC, CONN. (Baltic)—Elevation 140 feet
Record: November, 1911, to July, 1949

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.72	2.46	7.74	0.61	3.27	3.43	6.22	4.39	1.86	4.20	6.45	7.28	52.63
1943	3.28	2.56	3.73	4.24	5.03	2.08	2.64	1.88	0.44	5.08	4.15	1.70	36.81
1944	2.00	2.52	6.19	4.04	1.57	3.21	1.07	0.98	7.89	2.29	8.28	2.77	42.81
1945	2.64	3.61	2.47	3.76	4.54	3.35	4.13	2.42	1.62	2.05	5.61	6.90	43.60
1946	3.74	3.35	2.16	1.99	4.47	2.96	4.33	7.35	2.88	0.64	1.24	3.89	39.00
1947	2.89	2.17	2.84	4.81	2.09	3.52	3.62	2.57	5.25	2.47	5.65	3.69	41.57
1948	4.22	2.49	4.12	4.45	8.74	3.12	3.81	3.01	0.67	2.31	6.62	3.76	50.32
1949	4.68	2.48	3.62	—	3.94	0.02	2.36	—	—	—	—	—	—
1950	—	—	—	—	—	—	—	—	—	—	—	3.44	—
Av. 37 yrs.	3.60	2.57	3.91	3.71	3.46	3.39	3.90	3.66	3.75	3.09	3.58	3.68	42.30

* Continued from this JOURNAL 70: 3, 340, Sept., 1956.

BARKHAMSTEAD, CONN.—Elevation 450 feet
Record: February, 1923, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.66	3.43	6.22	0.81	5.01	5.18	5.65	2.75	4.88	3.95	6.54	5.83	52.91
1943	2.71	1.92	3.08	3.20	5.63	2.66	2.39	0.73	1.99	5.65	4.79	0.71	35.46
1944	1.86	2.10	4.76	4.66	2.12	3.44	2.76	3.98	7.06	2.22	6.74	4.19	45.89
1945	3.77	4.18	2.11	5.05	7.31	6.53	6.94	4.93	2.84	2.70	5.70	5.23	57.29
1946	3.28	3.04	1.71	2.35	6.28	2.88	7.19	3.79	4.32	1.62	0.87	2.69	40.05
1947	3.03	2.44	3.39	4.00	5.33	3.28	5.00	1.45	3.15	2.31	7.28	2.51	43.17
1948	3.11	1.98	3.43	3.32	6.23	6.34	2.64	1.13	0.63	2.38	4.89	8.74	44.82
1949	6.26	2.57	3.44	3.71	5.87	1.02	2.48	2.90	2.88	2.22	2.08	3.70	39.13
1950	4.39	4.54	2.98	2.83	3.95	5.60	2.37	4.17	1.54	2.00	7.52	5.95	47.84
Av. 27 yrs.	3.48	2.86	3.42	3.62	3.93	4.02	3.95	3.82	3.77	3.07	4.24	3.42	43.60

BLOOMFIELD, CONN.
Record: January, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	4.37	2.99	5.64	5.64	6.28	3.59	3.17	1.58	1.99	2.71	5.99	2.63	46.58
1941	2.60	3.11	2.24	1.05	2.81	4.79	4.59	2.39	1.08	1.39	3.52	4.04	33.59
1942	4.83	2.46	6.89	0.69	5.42	4.41	5.43	4.92	3.82	3.29	4.56	6.96	53.68
1943	3.26	2.11	4.21	3.27	4.92	3.05	2.95	1.86	1.00	5.90	4.19	0.87	37.59
1944	1.93	2.17	5.28	4.04	1.12	3.30	1.50	2.23	8.24	2.24	7.00	3.33	42.38
1945	3.59	3.82	2.44	5.36	6.03	3.39	5.20	4.41	2.84	2.15	5.19	6.80	51.22
1946	2.61	2.59	1.37	2.35	5.81	3.03	3.31	3.68	2.42	0.91	1.13	3.19	32.40
1947	3.07	2.16	2.77	4.80	4.20	4.17	5.18	3.66	2.65	2.38	6.04	2.31	43.39
1948	3.31	1.92	3.84	4.08	5.23	6.27	1.93	3.73	0.58	1.11	5.66	6.14	43.80
1949	5.00	3.19	1.50	3.71	4.99	0.54	2.22	2.75	2.67	1.98	2.21	2.80	33.56
1950	4.00	2.88	3.03	2.52	3.24	5.47	3.24	2.30	1.34	2.02	5.24	4.46	39.74
Av. 11 yrs.	3.51	2.67	3.56	3.41	4.55	3.82	3.52	3.05	2.60	2.37	4.61	3.96	41.63

BRIDGEPORT, CONN. (Bridgeport Hydraulic Co.)—Elevation 20 feet
Record: January, 1894, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.82	2.61	8.15	1.12	4.43	3.06	6.69	9.51	2.59	3.11	6.62	5.91	58.62
1943	3.67	2.06	4.76	2.79	5.47	4.64	2.64	1.42	1.98	5.57	4.39	1.69	41.08
1944	4.02	2.02	8.50	5.22	0.86	2.21	0.45	0.83	12.38	1.67	7.21	3.29	48.66
1945	2.55	3.75	2.91	4.53	5.98	2.30	6.99	4.43	3.67	2.05	5.94	5.80	50.90
1946	2.28	2.54	2.11	1.92	8.28	3.56	4.26	3.95	4.05	0.67	1.70	2.52	37.84
1947	2.55	2.22	3.22	4.62	4.36	4.57	3.60	2.81	2.48	2.79	7.35	3.58	44.15
1948	3.97	1.81	4.17	4.93	8.57	5.67	3.21	2.74	1.19	1.76	5.60	6.46	50.08
1949	7.00	3.54	2.30	4.15	6.89	0.06	2.51	3.86	4.64	2.08	1.60	3.00	41.63
1950	3.27	3.71	3.27	2.87	4.70	3.86	6.06	4.46	1.54	2.24	5.04	4.50	45.52
Av. 57 yrs.	3.94	3.60	4.21	3.89	3.93	3.54	4.18	4.43	3.86	3.58	3.79	3.93	46.88

BRISTOL, CONN. (Bristol Water Dept.)
Record: January, 1911, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.49	1.98	6.86	0.93	4.54	4.78	3.99	5.12	3.52	3.79	4.86	6.29	51.15
1943	3.24	1.64	4.05	3.34	7.32	4.01	3.38	1.15	1.71	6.93	5.36	1.10	43.23
1944	1.86	2.02	5.54	4.40	2.59	3.99	3.64	2.12	8.10	2.18	5.18	4.02	45.64
1945	2.88	4.76	2.54	5.36	7.33	3.26	4.94	5.09	4.61	1.67	5.84	5.93	54.21
1946	2.69	2.60	1.69	2.41	7.15	3.23	5.80	4.08	5.19	1.22	1,82	3.11	40.99
1947	2.55	2.41	3.23	5.35	3.76	5.07	6.63	3.28	3.64	1.92	7.76	3.07	48.67
1948	3.71	2.11	4.04	3.88	6.30	5.95	3.11	2.37	0.83	2.12	6.94	8.87	50.23
1949	5.96	3.95	1.29	4.52	5.20	0.83	2.78	3.89	3.98	2.19	1.82	3.54	39.95
1950	4.27	4.32	3.35	3.56	3.69	3.97	4.15	4.18	1.41	2.34	6.38	5.75	47.39
Av. 40 yrs.	3.56	3.00	3.79	3.81	4.18	4.07	4.36	4.25	4.13	3.40	4.09	3.61	46.25

BULLS BRIDGE DAM, CONN.
Record: January, 1943, to October, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1943	2.84	1.59	2.95	3.03	6.38	3.74	4.06	2.36	1.58	4.70	5.83	0.96	40.02
1944	1.61	2.11	4.56	4.24	1.99	4.05	3.33	1.82	4.62	2.21	4.70	3.65	38.88
1945	4.09	2.76	2.01	5.31	6.34	4.41	6.16	5.62	3.28	2.35	4.84	3.58	50.75
1946	1.58	1.79	2.35	2.08	4.81	3.06	7.57	2.97	4.66	1.92	1.29	1.84	35.92
1947	2.41	1.64	3.17	3.96	5.40	3.65	3.39	2.19	2.17	2.89	7.07	2.56	40.50
1948	3.20	1.75	3.13	3.68	6.18	5.97	2.56	2.40	0.13	1.38	4.15	5.36	39.89
1949	5.09	2.66	—	3.74	5.82	0.90	4.41	2.02	3.23	2.24	1.51	4.77	—
1950	3.65	3.61	2.65	2.03	4.29	6.67	3.82	6.56	1.70	1.68	—	—	—
Av. 6 yrs.	2.62	1.94	3.03	3.72	5.18	4.15	4.51	2.89	2.74	2.57	4.65	2.99	40.99

BURLINGTON, CONN. (Phelps Brook Dam, Hartford Water Works)—Elevation 497 feet
Record: January, 1918, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.85	5.01	7.65	0.72	4.99	6.52	5.65	4.58	3.77	3.76	7.87	6.99	60.36
1943	3.32	2.49	4.17	3.54	7.17	3.25	3.40	3.17	1.69	7.63	5.78	0.92	46.53
1944	1.68	2.09	5.13	4.85	1.30	3.77	3.28	5.14	7.57	2.27	6.56	4.79	48.43
1945	3.71	3.18	2.24	5.95	7.26	6.07	5.36	4.99	3.19	2.54	4.54	5.94	54.97
1946	2.95	2.43	1.89	2.66	6.41	3.12	6.97	4.66	4.48	1.85	1.36	2.65	41.43
1947	2.72	2.83	3.11	4.45	4.77	4.29	5.55	2.37	3.48	2.42	7.56	2.32	45.87
1948	2.96	1.97	4.01	4.17	6.90	6.96	2.99	1.88	0.24	2.39	6.75	6.93	48.15
1949	7.76	2.86	3.14	4.20	6.37	0.85	2.36	3.66	3.36	2.18	1.90	3.35	41.99
1950	4.18	3.74	3.26	3.52	3.84	7.66	2.72	5.35	2.19	2.23	6.29	5.83	50.81
Av. 33 yrs.	3.42	2.77	3.61	3.84	4.35	4.13	4.25	4.13	4.20	3.02	4.29	3.54	45.55

CANAAN, CONN. (Falls Village)—Elevation 600 feet
Record: May, 1889, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	1.86	2.30	6.14	0.79	5.40	4.94	5.64	2.79	3.21	3.34	5.81	4.18	46.40
1943	3.03	1.19	2.41	2.74	5.34	3.18	3.88	1.11	1.80	4.05	5.84	0.70	35.27
1944	1.53	1.78	3.02	3.56	2.70	7.05	2.51	2.92	3.21	1.87	3.69	2.69	36.51
1945	3.71	1.89	1.88	5.58	6.67	6.71	8.22	3.65	4.89	2.31	3.81	3.35	52.67
1946	1.69	2.52	1.57	2.07	4.52	3.22	5.26	2.11	3.65	2.10	0.94	2.15	31.80
1947	2.47	2.49	3.26	4.11	5.05	3.54	3.60	0.93	3.39	1.65	5.32	2.04	37.85
1948	2.55	1.92	3.75	3.50	5.30	5.85	2.80	2.30	0.90	1.76	4.21	6.43	41.27
1949	6.94	2.38	1.32	3.54	5.55	0.93	2.99	2.77	2.04	2.75	1.50	4.10	36.81
1950	3.22	2.80	2.22	2.43	4.29	5.22	3.03	4.72	1.32	1.32	7.03	3.75	41.05
Av. 61 yrs.	3.06	2.92	3.27	3.23	3.91	4.06	4.38	3.95	4.16	3.12	3.31	3.25	42.62

COLCHESTER, CONN.—Elevation 460 feet
Record: October, 1891, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	2.54	3.71	4.45	4.36	4.70	4.25	3.89	4.35	3.80	3.75	4.20	4.16	48.16
1942	3.19	3.00	7.18	0.67	2.54	2.60	4.57	3.96	2.18	3.72	6.29	5.60	45.50
1943	2.96	1.83	4.19	3.34	4.14	3.20	3.04	1.49	0.65	4.74	5.12	1.06	35.76
1944	1.74	2.35	5.11	3.60	1.07	2.95	2.22	1.57	8.02	2.03	8.61	2.75	42.08
1945	3.01	3.47	1.98	4.44	5.97	4.15	3.55	2.68	1.90	2.19	5.96	6.80	46.10
1946	3.13	2.45	2.02	2.77	5.89	3.07	4.57	6.09	3.34	0.84	1.65	3.17	38.99
1947	2.39	2.26	2.80	4.62	2.87	3.64	3.67	4.05	3.39	1.51	5.56	3.19	39.95
1948	4.12	2.20	—	4.51	—	—	—	—	—	—	—	—	—
1949	—	—	—	—	—	—	—	—	—	—	—	—	—
1950	—	2.41	3.13	2.84	5.03	2.81	2.15	5.64	1.49	1.53	5.16	4.28	—
Av. 56 yrs.	3.96	3.66	4.38	4.34	3.87	3.47	3.89	4.38	3.78	3.65	4.18	4.12	47.68

COLLINSVILLE, CONN. (Canton)—Elevation 300 feet
Record: January, 1929, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.93	4.30	7.68	0.88	4.43	5.80	5.13	4.52	3.31	3.25	6.83	6.66	55.72
1943	3.82	2.15	4.19	3.62	6.37	3.30	2.97	2.63	1.52	6.52	5.13	0.95	43.17
1944	2.14	2.49	5.61	4.18	1.26	3.51	2.65	3.91	7.24	2.01	6.30	4.18	45.48
1945	4.21	4.23	2.93	5.51	7.11	5.93	4.83	4.22	2.29	1.74	6.07	5.81	54.88
1946	2.51	2.83	1.71	2.59	6.49	3.06	6.13	5.32	4.32	1.60	1.36	2.86	40.78
1947	3.04	2.73	3.41	4.20	5.02	4.67	5.30	2.55	3.39	2.47	6.35	3.07	46.20
1948	3.82	2.19	4.16	4.01	6.49	6.23	2.95	1.79	0.27	3.00	7.24	7.03	49.18
1949	7.02	3.00	3.35	3.91	6.09	0.93	2.14	3.07	3.18	2.11	1.95	3.30	40.05
1950	4.50	4.78	3.48	3.54	3.55	7.50	2.75	4.88	2.21	2.06	5.74	5.87	50.86
Av. 22 yrs.	4.03	3.06	4.14	3.91	4.37	4.36	3.90	4.00	3.95	2.98	4.23	3.83	46.76

CORNWALL, CONN. (Cream Hill)—Elevation 1,300 feet
Record: June, 1896, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.73	2.36	6.18	0.84	5.39	5.11	6.61	2.34	3.62	3.53	7.00	5.74	51.45
1943	3.40	2.15	2.66	3.30	5.86	2.61	3.92	1.34	2.25	4.83	6.47	0.98	39.77
1944	2.01	2.71	5.25	3.75	2.34	6.98	2.56	1.78	4.39	1.73	5.27	3.23	42.00
1945	4.81	4.32	2.02	5.36	7.06	10.05	9.10	4.14	5.35	2.83	4.42	4.80	64.26
1946	1.82	3.69	1.70	2.26	4.40	3.02	4.61	3.81	4.33	1.88	1.35	1.63	34.50
1947	2.55	3.85	3.05	3.54	5.22	4.00	3.70	1.83	3.93	1.61	5.25	2.38	40.91
1948	3.67	1.55	4.05	3.32	4.97	5.52	0.48	2.96	0.86	1.59	4.68	9.52	43.17
1949	4.72	3.17	2.79	3.58	4.79	0.89	4.56	2.76	3.56	2.32	1.54	4.37	39.05
1950	3.85	2.91	2.50	2.85	4.10	5.68	3.68	4.75	1.71	2.35	4.89	4.48	43.75
Av. 54 yrs.	3.57	3.26	3.67	3.51	4.13	4.36	4.57	4.08	4.20	3.38	3.70	3.64	46.07

DANBURY, CONN.—Elevation 390 feet
Record: January, 1926, to December, 1935; November, 1937, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.44	2.93	6.35	0.93	4.91	2.60	5.81	—	—	3.16	7.09	5.63	—
1943	3.14	1.92	3.54	3.76	—	—	4.16	2.10	1.49	4.52	—	1.01	—
1944	—	—	5.00	4.85	1.49	4.35	4.70	—	—	—	—	3.73	—
1945	—	—	—	—	—	—	—	—	4.41	2.40	5.86	5.97	—
1946	1.98	2.49	2.16	1.70	6.84	2.90	4.53	6.27	5.38	1.43	1.53	2.19	39.40
1947	2.53	3.33	3.71	4.03	6.29	5.00	2.94	5.46	3.39	1.87	8.56	3.85	50.96
1948	3.67	1.55	4.05	3.32	4.97	5.52	0.48	2.96	0.86	1.59	4.68	9.52	43.17
1949	4.94	2.80	1.78	3.59	5.75	1.08	2.23	5.97	2.70	1.96	1.28	3.38	37.46
1950	2.88	3.09	2.64	2.88	4.16	3.02	7.71	2.91	1.24	2.11	7.04	5.49	45.17
Av. 18 yrs.	3.11	2.85	3.27	3.58	4.21	3.83	4.40	4.65	4.12	2.84	4.06	3.22	44.14

DAWSON LAKE, CONN. (Formerly Woodbridge)—Elevation 170 feet
Record: November, 1913, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.45	4.61	8.23	0.90	4.25	2.79	6.75	7.85	2.76	3.45	4.76	6.21	55.01
1943	2.89	2.05	5.21	3.14	4.31	3.97	2.93	1.38	1.99	6.19	5.20	1.48	40.74
1944	2.18	2.30	8.23	4.31	3.79	1.76	0.45	1.30	10.84	1.96	6.87	4.19	48.18
1945	3.20	4.23	2.48	4.61	7.06	2.50	4.77	3.54	3.41	2.25	5.98	6.02	50.05
1946	2.34	2.99	1.66	2.02	6.58	2.74	4.12	7.81	3.41	1.64	1.38	2.37	39.06
1947	1.90	2.76	3.30	4.40	4.38	4.49	6.92	3.43	2.21	2.00	7.40	3.92	47.11
1948	4.39	2.28	4.93	5.30	7.97	4.52	3.91	3.53	0.17	1.96	6.65	7.04	52.65
1949	7.89	3.02	2.94	4.68	6.52	0.23	3.34	2.69	4.31	1.90	2.03	2.93	42.48
1950	3.43	3.54	3.04	3.27	4.61	3.78	5.15	5.92	1.00	2.30	5.06	5.16	46.26
Av. 37 yrs.	3.59	3.03	3.85	3.90	4.07	3.91	3.97	4.08	3.45	3.18	3.92	3.78	44.73

EAST HAVEN, CONN. (Lake Saltonstall)—Elevation 25 feet
Record: November, 1913, to December, 1949

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.30	3.43	6.66	0.95	2.90	2.77	5.95	9.65	1.90	3.63	4.65	5.51	50.30
1943	3.45	1.95	3.73	2.19	4.20	3.63	3.46	1.42	0.98	4.00	3.54	1.69	34.24
1944	2.89	2.36	6.59	4.65	0.93	1.59	1.10	1.21	8.69	1.78	7.66	3.11	42.56
1945	2.21	3.49	2.76	4.01	5.05	2.55	5.23	5.85	2.85	2.43	5.18	5.75	47.36
1946	2.68	2.47	1.68	1.69	7.16	3.28	4.34	7.12	2.61	1.39	1.77	2.81	39.00
1947	2.33	2.02	2.23	4.56	4.17	4.13	4.05	4.91	3.51	1.84	8.21	2.87	44.83
1948	4.69	1.61	4.68	5.67	7.64	4.30	2.49	2.66	0.15	2.11	5.32	4.25	45.57
1949	6.77	2.98	3.05	3.25	6.22	0.11	3.19	2.06	4.36	2.40	2.30	3.05	39.74
1950	—	—	—	—	—	—	—	—	—	—	—	—	—
Av. 36 yrs.	3.44	2.95	3.57	3.60	3.62	3.72	3.62	3.79	3.45	2.96	3.52	3.59	41.83

EASTON LAKE RESERVOIR, CONN. (Easton)—Elevation 313 feet
Record: October, 1922, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.18	4.14	8.33	0.75	4.42	4.02	6.93	9.30	2.87	3.43	7.14	5.44	59.95
1943	2.97	1.95	7.33	3.67	6.68	5.29	3.26	1.42	2.30	6.23	4.42	1.44	46.96
1944	3.05	2.15	8.71	4.91	1.39	1.92	0.41	1.52	10.71	1.79	6.57	3.34	46.47
1945	—	—	—	—	—	—	7.42	3.69	4.39	—	5.81	5.68	—
1946	2.21	2.59	1.92	1.79	8.55	3.31	4.34	4.77	5.95	1.10	1.88	2.22	40.63
1947	2.60	1.84	3.84	4.63	5.15	4.22	3.58	4.91	2.96	2.81	7.52	3.19	47.25
1948	4.05	1.73	4.46	5.58	7.50	3.53	3.57	2.42	0.69	1.46	5.85	6.71	47.55
1949	7.36	2.97	2.48	4.51	5.64	0.15	2.36	2.40	4.20	2.08	1.54	2.66	38.35
1950	2.99	3.90	3.27	2.84	4.42	—	4.65	3.29	1.58	2.74	5.65	4.74	—
Av. 22 yrs.	3.79	3.09	4.18	4.13	4.35	3.77	3.46	4.82	4.66	3.18	3.91	3.34	46.68

GROTON, CONN. (Poquonock Bridge)—Elevation 16 feet
Record: January, 1931, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1943	4.24	2.68	3.30	3.22	4.02	2.63	4.21	1.17	0.37	4.07	2.87	1.71	34.49
1944	2.66	2.69	5.95	4.17	0.78	2.94	0.82	1.53	7.38	1.76	9.12	3.54	43.34
1945	3.54	4.45	2.43	4.02	4.07	2.48	1.57	3.89	1.39	2.70	6.11	7.88	44.53
1946	3.95	3.06	1.72	2.09	4.69	3.13	2.15	8.26	2.31	0.66	1.01	2.36	35.37
1947	3.19	2.11	2.49	5.86	3.88	2.64	2.99	3.67	2.89	2.51	7.24	3.72	43.19
1948	9.65	2.83	5.34	3.86	8.20	3.09	1.79	0.73	0.96	3.59	5.01	3.79	48.84
1949	6.51	4.06	2.89	3.95	3.08	0.03	3.62	2.66	3.75	2.15	4.27	4.24	41.21
1950	3.84	3.84	3.25	2.37	6.46	3.72	2.47	4.34	0.88	1.81	4.03	4.25	41.26
Av. 19 yrs.	4.69	3.67	4.24	3.94	3.72	3.51	3.24	3.71	3.58	2.94	4.29	3.88	45.41

HAMPDEN, CONN. (Lake Whitney, New Haven Water Co.)—Elevation 40 feet
Record: November, 1913, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.71	3.61	8.64	1.04	3.95	3.15	7.52	10.21	2.27	4.20	5.09	6.26	58.65
1943	3.61	2.05	5.33	3.11	4.66	3.69	3.86	1.26	1.36	5.17	4.15	1.60	39.85
1944	2.83	2.07	8.91	4.79	2.93	2.14	0.52	1.17	11.61	2.25	7.78	4.42	51.42
1945	2.59	4.06	2.43	4.35	6.98	2.46	5.18	3.84	3.01	2.63	5.56	6.30	49.39
1946	2.72	2.63	1.65	2.23	6.75	2.95	3.32	8.07	3.61	1.41	1.71	3.19	40.24
1947	2.99	2.40	3.18	4.67	3.87	4.31	4.44	2.95	2.39	2.47	9.16	3.30	46.13
1948	4.86	2.02	5.45	5.61	8.73	4.67	3.85	2.79	0.18	2.45	6.77	6.55	53.93
1949	7.72	3.25	4.00	4.10	7.05	0.16	3.95	2.76	4.47	2.37	2.09	3.12	45.04
1950	3.90	3.69	3.31	2.23	4.19	3.60	4.30	5.83	1.13	1.79	4.99	4.76	43.72
Av. 37 yrs.	3.78	3.31	4.09	3.94	4.03	4.05	4.04	4.26	3.54	3.16	3.96	4.01	46.17

HARTFORD, CONN. (U.S. Weather Bureau, Airport)
Record: April, 1898, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.20	2.11	6.13	0.65	3.89	4.03	4.47	4.31	3.24	2.80	7.05	6.69	49.57
1943	3.75	2.02	4.54	2.99	5.31	3.48	3.27	1.18	0.65	4.82	4.32	0.91	37.24
1944	1.51	1.85	5.02	4.55	1.23	3.81	2.26	2.44	8.79	2.38	7.14	3.07	44.05
1945	3.75	4.55	2.52	4.67	6.01	4.53	3.31	3.83	1.59	2.09	5.81	6.85	49.51
1946	2.72	2.81	1.84	2.58	5.87	2.99	4.84	6.76	2.36	0.90	1.28	2.84	37.79
1947	3.18	2.61	2.09	5.20	3.10	4.76	6.32	3.31	2.97	2.10	5.23	3.55	44.42
1948	4.17	1.96	3.74	3.85	4.81	7.79	2.37	2.63	0.63	1.46	6.84	5.50	45.75
1949	4.99	3.32	2.37	3.78	5.63	0.87	3.23	3.02	3.03	2.06	2.74	3.40	38.44
1950	3.87	4.41	2.63	3.06	2.82	3.85	4.09	4.16	1.09	1.85	4.83	4.12	40.78
Av. 52 yrs.	3.57	3.37	3.85	3.66	3.61	3.62	3.97	3.94	3.57	2.94	3.45	3.69	43.24

HARTFORD, CONN.
Record: November, 1940, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	—	—	—	—	—	—	—	—	—	—	5.65	2.62	—
1941	2.53	2.33	2.42	1.09	4.24	4.15	6.03	2.71	0.67	1.61	3.35	3.31	34.44
1942	4.57	2.17	6.28	0.68	4.28	4.59	4.21	4.83	3.58	3.07	6.79	6.68	51.73
1943	3.83	1.81	4.23	3.03	5.29	3.78	3.22	1.32	0.69	5.00	4.08	0.85	37.13
1944	1.61	2.03	4.89	4.33	1.12	3.83	2.60	2.38	8.13	2.63	6.86	3.03	43.44
1945	3.69	4.05	2.47	5.09	6.06	3.98	4.42	3.57	1.83	1.98	5.50	6.04	48.68
1946	2.45	2.56	1.78	2.64	6.15	2.80	4.83	5.36	2.40	0.78	1.27	3.17	36.19
1947	2.82	2.58	2.28	4.91	3.39	4.67	5.85	2.96	2.75	1.96	5.65	2.86	42.68
1948	3.50	1.85	3.99	3.88	4.69	7.69	1.94	3.46	0.44	1.08	6.59	5.21	44.32
1949	4.99	2.79	2.37	3.73	4.86	0.92	2.42	2.97	2.93	1.85	2.23	2.22	34.28
1950	3.79	3.65	2.65	2.50	2.99	4.88	3.04	4.30	0.96	1.71	4.83	3.79	39.09
Av. 10 yrs.	3.38	2.58	3.34	3.19	4.31	4.13	3.86	3.39	2.44	2.17	4.71	3.72	41.22

HEMLOCKS RESERVOIR, CONN. (Fairfield)—Elevation 253 feet
Record: October, 1922, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	3.12	3.81	8.32	1.05	3.87	3.73	7.33	11.02	2.62	2.87	6.43	5.24	59.41
1943	3.03	1.89	4.61	3.60	6.45	4.87	2.56	1.35	2.39	6.14	3.85	1.41	42.15
1944	3.09	1.84	8.68	4.60	1.73	1.19	0.43	1.31	10.83	1.69	6.48	3.16	45.03
1945	—	—	—	—	—	—	8.19	4.22	4.04	—	5.53	6.33	—
1946	2.15	2.40	2.18	1.88	8.10	4.06	4.61	4.13	5.55	0.88	1.87	2.21	40.02
1947	2.63	2.00	3.62	4.80	5.20	4.71	3.42	2.76	2.93	2.62	7.57	3.99	46.25
1948	3.91	1.56	3.78	5.78	8.44	5.26	2.88	2.93	1.09	1.45	5.83	7.01	49.92
1949	6.95	—	—	4.60	6.18	0.10	2.32	3.67	3.97	1.90	1.43	2.98	—
1950	3.22	3.93	3.43	2.50	—	—	5.23	3.83	1.55	2.94	5.45	4.93	—
Av. 24 yrs.	3.80	2.98	4.28	3.99	4.02	3.86	3.52	4.81	4.32	3.46	4.28	3.58	46.90

LAKE KONOMOC, CONN. (New London Water Works)
Record: 1885 to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	6.04	4.17	4.55	0.96	2.51	2.87	5.52	6.21	1.66	5.26	6.99	8.77	55.51
1943	2.97	2.85	4.24	4.09	5.42	3.22	4.36	1.61	0.50	4.58	3.88	2.38	40.10
1944	2.81	2.62	7.46	5.17	1.16	4.91	1.29	2.30	12.26	3.97	3.70	3.53	51.18
1945	2.34	3.69	—	—	5.40	3.15	2.85	3.63	2.28	2.05	5.85	6.73	—
1946	3.66	3.54	2.00	1.83	5.20	4.01	3.23	6.78	1.81	2.49	1.56	2.77	38.88
1947	2.76	1.25	2.64	4.76	3.49	3.01	3.41	4.49	2.72	2.92	7.65	1.84	40.94
1948	4.73	—	5.24	3.64	10.21	3.60	1.92	1.27	—	3.62	5.79	—	—
1949	5.46	4.06	—	6.37	4.16	0.04	2.77	3.28	5.85	2.28	3.24	3.27	—
1950	—	3.98	3.44	—	7.42	3.18	2.08	5.27	0.99	3.52	4.54	3.96	—
Av. 52 yrs.	4.26	4.05	4.46	4.22	3.56	3.46	3.70	4.43	3.70	3.92	3.73	4.22	47.71

LAUREL RESERVOIR, CONN. (Stamford)
Record: January, 1927, to November, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.66	2.44	8.31	1.48	4.60	3.22	5.54	10.69	3.93	4.57	3.70	5.22	58.36
1943	3.65	2.16	3.38	3.43	5.40	3.43	4.50	1.80	2.51	6.88	3.58	1.32	42.04
1944	3.17	2.46	8.26	4.77	2.21	2.79	0.54	1.28	9.64	2.85	6.31	4.38	48.66
1945	—	—	—	—	—	—	10.23	—	3.94	—	5.03	6.14	—
1946	2.00	2.88	2.39	1.78	11.11	3.57	5.72	7.64	6.58	1.30	1.72	2.34	49.03
1947	2.72	2.94	4.26	5.51	5.50	5.10	3.81	2.05	3.48	1.53	7.66	4.47	49.03
1948	4.19	1.69	5.48	3.65	7.41	4.98	3.71	3.22	0.23	2.14	4.79	6.03	47.52
1949	6.53	2.93	3.56	4.23	5.71	0.23	3.46	3.52	3.40	1.64	1.24	3.33	39.78
1950	2.90	4.37	3.36	2.35	4.34	—	5.33	4.50	1.33	1.46	5.14	—	—
Av. 22 yrs.	3.89	3.04	4.40	3.92	4.61	4.10	4.13	5.14	4.51	3.28	3.80	3.56	48.38

MANCHESTER, CONN.
Record: January, 1938, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1938	5.26	2.39	1.68	1.98	3.93	5.43	9.10	4.00	17.46	1.09	3.33	2.93	58.58
1939	2.99	4.90	3.60	4.19	1.07	3.48	2.93	5.04	2.91	4.73	1.50	3.91	41.25
1940	3.17	2.45	5.70	5.54	5.88	4.13	3.82	1.64	3.26	2.28	5.47	2.72	46.06
1941	1.97	2.84	2.75	1.05	2.88	5.56	5.49	2.94	0.30	3.17	2.15	3.72	34.82
1942	4.38	0.99	6.01	0.27	3.81	3.94	5.55	4.04	2.52	3.51	5.28	6.83	47.13
1943	2.13	1.38	2.98	—	3.97	3.08	3.46	1.89	1.06	4.55	4.03	0.85	—
1944	1.82	1.84	4.33	4.52	1.57	5.40	1.71	1.90	9.46	2.40	5.56	3.73	44.24
1945	2.87	2.99	2.28	4.14	5.65	4.35	3.74	2.81	1.33	2.64	5.43	5.52	43.75
1946	2.88	2.48	1.65	2.62	4.88	2.94	5.12	5.12	1.67	1.80	1.77	2.56	35.49
1947	2.92	2.30	1.93	3.97	2.95	4.23	6.03	3.48	4.01	1.58	5.32	2.85	41.57
1948	4.41	1.87	3.98	3.19	5.50	5.48	2.80	2.08	0.21	1.74	5.69	3.34	40.29
1949	5.53	2.80	2.96	3.58	5.90	0.25	2.39	1.86	2.71	1.76	2.04	3.38	35.16
1950	3.51	3.73	3.20	3.52	3.36	4.66	3.15	5.12	1.54	1.73	5.26	4.41	43.19
Av. 12 yrs.	3.47	2.63	3.34	3.21	3.95	4.15	4.32	3.34	3.95	2.37	4.07	3.83	42.63

MEAD POND RESERVOIR, CONN. (Formerly New Canaan)—Elevation 485 feet
Record: October, 1928, to September, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	4.61	2.50	8.49	1.28	4.71	3.98	6.11	12.61	4.74	4.92	4.03	5.83	63.81
1943	3.28	2.20	4.64	3.79	5.85	4.19	3.71	1.90	2.77	6.56	4.40	1.38	44.67
1944	2.92	2.77	8.26	4.93	1.88	2.63	0.66	1.45	10.03	3.00	6.98	4.23	49.74
1945	—	—	—	—	—	—	9.87	5.00	4.20	—	5.90	5.96	—
1946	2.39	2.75	2.44	1.73	10.43	3.46	5.56	5.82	6.74	1.35	1.83	2.27	46.77
1947	2.88	3.46	4.06	5.42	5.20	4.74	5.03	3.53	2.95	2.11	8.84	4.52	52.74
1948	4.17	1.67	5.79	3.82	7.49	4.76	4.19	2.74	0.24	2.49	5.02	6.56	48.94
1949	6.70	3.22	3.75	4.46	5.74	0.12	3.26	3.22	3.45	1.84	1.44	3.75	40.95
1950	3.17	4.60	3.73	2.82	4.44	6.35	4.30	1.65	2.17	—	—	—	—
Av. 20 yrs.	4.03	3.06	4.68	4.09	4.83	4.18	3.99	4.80	4.67	3.31	3.98	3.67	47.29

MIDDLETOWN, CONN.—Elevation 125 feet
Record: September, 1858, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	2.60	2.65	2.64	—	—	—	—	—	—	—	—	—	—
1942	3.22	3.91	7.90	0.80	3.33	2.83	6.10	6.22	—	—	—	—	—
1943	3.87	—	4.62	3.56	5.82	4.58	3.34	2.52	0.68	6.44	5.84	1.28	—
1944	1.86	2.66	6.62	5.79	1.50	4.55	—	0.92	12.40	2.83	8.45	3.05	—
1945	3.09	4.01	2.39	—	—	—	—	—	—	—	—	—	—
1946	2.61	2.60	2.05	2.95	8.00	3.60	4.56	7.05	2.49	1.13	1.94	3.05	42.03
1947	3.04	2.37	2.86	4.97	4.50	5.18	4.31	3.06	3.52	3.13	6.70	2.76	46.39
1948	4.62	1.84	4.35	4.82	5.76	5.07	4.47	2.07	0.32	2.31	7.01	4.72	47.36
1949	6.42	2.65	2.57	4.88	6.69	0.86	3.69	3.31	4.69	1.98	2.20	3.20	43.14
1950	3.33	4.42	3.24	3.18	4.19	3.53	3.16	4.68	1.24	2.07	6.68	4.50	44.22
Av. 87 yrs.	4.01	3.73	4.31	3.74	3.95	3.61	4.00	4.47	3.70	3.79	3.96	3.89	47.16

MILFORD, CONN. (New Haven Water Co.)—Elevation 35 feet
Record: January, 1926, to August, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1942	2.63	3.71	7.93	1.13	3.98	4.15	7.28	8.92	2.59	2.74	5.38	5.90	56.34
1943	3.18	2.24	4.36	2.42	5.34	4.70	1.94	1.69	1.41	5.34	3.79	1.94	38.35
1944	3.38	1.78	8.05	5.37	0.86	1.25	1.13	1.24	10.61	2.28	6.56	3.42	45.93
1945	2.12	3.73	3.11	3.93	6.23	2.57	4.41	4.70	2.70	2.03	4.42	5.58	45.53
1946	2.49	2.26	1.98	1.85	7.16	3.86	2.60	6.34	3.68	0.85	1.67	2.96	37.70
1947	2.76	1.47	2.83	4.72	4.31	3.71	5.75	3.32	3.14	2.92	6.35	2.87	44.15
1948	4.24	1.76	4.55	4.61	8.93	4.59	4.04	2.76	0.49	2.23	6.18	6.10	50.48
1949	5.85	3.19	2.87	3.77	6.52	0.18	2.89	3.64	4.32	2.22	1.82	3.00	40.27
1950	—	—	3.12	3.10	—	—	—	5.91	—	—	—	—	—
Av. 24 yrs.	3.47	3.02	4.15	3.89	3.72	3.97	3.81	4.30	3.95	3.07	3.96	3.67	44.98

MOHAWK FOREST RANGER STATION, CONN.
Record: April, 1941, to December, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1941	—	—	—	0.80	—	5.95	6.42	2.57	0.84	2.67	3.03	3.77	—
1942	1.87	3.21	6.08	1.01	4.85	5.28	5.89	3.03	3.67	4.08	6.93	4.99	50.89
1943	3.48	1.71	—	3.54	6.82	4.86	3.85	2.02	2.39	4.77	6.57	0.77	—
1944	1.41	2.27	4.71	3.75	1.65	7.85	—	1.73	5.42	—	5.16	2.97	—
1945	4.78	3.70	2.08	5.75	7.60	—	8.35	5.41	3.89	—	4.75	4.60	—
1946	2.38	2.64	2.14	2.59	5.59	3.39	6.09	3.49	4.43	2.74	1.19	2.24	38.91
1947	3.05	3.05	4.09	3.83	5.65	4.38	3.91	3.94	2.71	1.22	6.97	3.27	46.07
1948	3.31	2.08	3.62	3.87	5.44	7.01	3.25	2.97	0.63	1.77	4.84	8.64	47.43
1949	8.15	2.66	3.21	3.58	6.14	0.72	5.19	2.96	3.61	2.46	1.68	4.29	44.65
1950	4.02	3.89	3.38	2.68	4.69	5.24	2.75	5.20	1.47	1.95	8.53	4.79	48.59
Av. 6 yrs.	3.80	2.92	3.75	2.93	5.39	4.34	4.51	3.60	2.75	2.37	5.02	4.70	46.08

MOODUS RESERVOIR, CONN. (East Haddam)
Record: January, 1940, to November, 1950

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	2.89	—	4.40	5.28	6.91	1.84	4.27	—	—	1.72	5.26	2.66	—
1941	2.94	3.11	2.58	1.09	2.67	3.75	3.09	3.67	0.33	1.95	3.30	4.42	32.90
1942	4.78	2.42	8.47	0.76	2.21	0.31	4.14	3.80	2.37	3.75	6.20	7.23	46.44
1943	3.33	2.17	4.18	3.52	3.74	3.85	3.53	1.18	0.53	4.67	3.35	1.40	35.45
1944	2.76	2.61	6.75	4.50	1.24	4.19	3.11	1.33	8.83	2.19	8.78	3.55	49.84
1945	2.77	3.92	2.25	4.71	5.67	2.60	3.50	3.06	2.03	1.90	6.19	7.09	45.69
1946	2.80	3.28	2.10	2.00	5.87	3.60	3.98	5.83	3.33	0.62	1.66	3.29	38.36
1947	2.80	1.79	3.27	—	1.65	4.00	4.32	4.82	3.10	2.02	7.18	3.42	—
1948	4.14	1.85	4.04	4.73	8.24	3.72	4.97	2.02	0.77	1.89	6.14	5.50	48.01
1949	5.89	3.15	2.28	4.39	4.48	0.37	2.31	5.27	2.47	—	2.20	2.52	—
1950	3.51	4.32	3.16	2.59	5.09	3.27	—	4.57	1.52	1.41	5.17	—	—
Av. 7 yrs.	3.36	2.77	4.34	3.04	4.23	3.15	3.76	2.98	2.60	2.42	5.09	4.64	42.38

PROCEEDINGS

NOVEMBER 1956 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, NOVEMBER 15, 1956

President Edward L. Tracy in the Chair.

The President announced the death on November 10, 1956, of Fred P. Stradling, Manager of the Bristol County Water Co., Bristol, R. I., who had been a member of the Association since February, 1947.

Secretary Knox announced the election of the following to membership in the Association:

John W. L. White, Engineer, Consumers Water Co., Portland, Me.; Harry M. Wooster, Engineer, Consumers Water Co., Portland, Me.; William T. Sherman, Superintendent, Water Department, Scituate, Mass.

The membership and organization of the permanent Scholarship Committee were announced by President Tracy and William B. Duffy as follows:

William B. Duffy, *Chairman*
Thomas R. Camp, *Vice-Chairman*
Richard M. Cleveland, *Secretary*
William P. Melley, *Treasurer*
Richard N. Berry
Roger W. Esty
Kenneth F. Knowlton

Edward C. Monahan, Town Manager, Arlington, Mass., gave a talk on "Municipal Administration and Finance."

A paper on "Control of New England Floods" was read by Brigadier General Robert J. Fleming, Jr., Division Engineer, New England Division, Corps of Engineers, U. S. Army, Boston, Mass.

DECEMBER 1956 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, DECEMBER 20, 1956

President Edward L. Tracy in the Chair.

The President announced the death on November 16, 1956, of Frank J. Thiery, Superintendent of Water Department, Bristol, Conn., who had been a member of the Association since February, 1943.

Secretary Knox announced the election of the following to membership in the Association:

Harold Bateson, Consulting Engineer, Lakewood, R. I.; Alfred F. Bridgman, Assistant Civil Engineer, Water Division, Metropolitan District Commission, Clinton, Mass.; Norman E. Jackson, Engineer, Camp, Dresser & McKee, Boston, Mass.; Irving J. LaMay, President, LaMay Construction Co., Old Saybrook, Conn.; Richard W. LaMay, Superintendent, LaMay Construction Co., Old Saybrook, Conn.; Myrl R. Limeburner, Acting Superintendent, Water Department, Bristol, Conn.; Charles C. Manganaro, Consulting Engineer, Beechhurst, N. Y.; Vito F. Pennacchio, Assistant Project Engineer, Coffin & Richardson, Inc., Boston, Mass.

A paper on "Use of the Magnetic Flow Meter at Medfield, Mass.," was read by Edwin B. Cobb, Partner, Metcalf & Eddy, Boston, Mass.

A paper on "History and Operating Principle of the Magnetic Flow Meter" was read by Russell H. Babcock, Manager, Utilities Industries Division, The Foxboro Co., Foxborough, Mass.

A motion picture on the operation of the Gradall in municipal work was shown by the courtesy of Hedge & Mattheis Co., Boston, Mass. James O. Jordan, District Manager, Hedge & Mattheis Co., read a short paper.

JANUARY 1957 MEETING

HOTEL STATLER, BOSTON, MASS.

THURSDAY, JANUARY 17, 1957

President Edward L. Tracy in the Chair.

Secretary Knox announced the election of the following new member of the Association:

Ralph I. Larson, Director of Public Works, Natick, Mass.

R. John Griefen, Vice-President, R. M. Bradley & Co., Inc., Boston, Mass., gave a talk on "New England's Water Supply and Our Industrial Growth."

A paper on "Chlorine and Cylinder Handling" was read by Richard A. Knef, Fields Point Manufacturing Corp., Providence. R. I.

American Water Works Association

Tentative

Standard Specifications

for

DIMENSIONS FOR STEEL WATER PIPE FITTINGS

The American Water Works Association has adopted and promulgates these "Dimensions for Steel Water Pipe Fittings." They are to be considered as dimensional only. They are based upon the best known experience and are intended for use under normal conditions. They are not designed for unqualified use under all conditions and the advisability of use of the dimensions herein specified for any installation must be subjected to review by the engineer responsible for the construction in the particular locality concerned.

Approved as "Tentative" Jul. 14, 1955

Third Printing, March 1957

AMERICAN WATER WORKS ASSOCIATION
Incorporated
2 Park Avenue, New York 16, N.Y.

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Acknowledgment is made and appreciation is expressed to the Manufacturers' Technical Advisory Committee to Committee 8310 D for its assistance and cooperation in the preparation of these specifications.

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Tentative Standard Specifications for

Dimensions for Steel Water Pipe Fittings

Scope. This standard covers overall dimensions for steel water pipe fittings. It is not intended to cover thickness, shape, welding, attachments, and tolerance of fittings.

Size. The "size" of the fittings in the following tables is identified by the corresponding "nominal pipe size," and may be either inside diameter or outside diameter, depending upon the purchaser's requirements.

TABLE 1 (*See Fig. 1*)

Dimensions for Steel Water Pipe Fittings, Class 125
for Service in Piping Systems in Water Works or Industrial Plants *
(*All Dimensions in Inches*)

Nominal Diam.	Tee		Cross $A+A$	Lateral (45–75 deg)		True Wye (45 deg)		Elbows					Reducer Cone G
	Run $A+A$	Outlet A		Run D	Outlet E	E'	F'	Two-Piece (0–30 deg) C	Three-Piece (31–60 deg) C	Four-Piece (61–90 deg) A	Four-Piece Long-Radius B	Smooth (90 deg) T	
4	13	6.5	13	15	12	6.5	3	4	4	6.5	9	6	7
6	16	8	16	18	14.5	8	3.5	5	5	8	11.5	9	9
8	18	9	18	22	17.5	9	4.5	5.5	5.5	9	14	12	11
10	22	11	22	25.5	20.5	11	5	6.5	6.5	11	16.5	15	12
12	24	12	24	30	24.5	12	5.5	7.5	7.5	12	19	18	14
14	28	14	28	33	27	14	6	7.5	7.5	14	21.5	21	16
16	30	15	30	36.5	30	15	6.5	8	8	15	24	24	18
18	33	16.5	33	39	32	16.5	7	8.5	8.5	16.5	26.5	27	19
20	36	18	36	43	35	18	8	9.5	9.5	18	29	30	20
22	40	20	40	46	37.5			10	10	20	31.5	33	22
24	44	22	44	49.5	40.5	22	9	11	11	22	34	36	24
30	50	25	50	59	49	25	10	15	15	25	41.5	45	30
36	56	28	56	84†	60†			18	18	28	49		36
42	62	31	62	95†	69†			21	21	31	56.5		42
48	68	34	68	104†	77†			24	24	34	64		48

* All dimensions shown conform to the dimensions given in ASA B16.1 for cast-iron pipe flanges and flanged fittings.

† These fittings shall conform to AWWA Standard Specifications for Cast-Iron Pressure Fittings, C100. If these are to be used with mechanical couplings, add 8 in. to each end for diameters up to 30 in.; 10 in. to each end for diameters of 30 in. and more. For laterals, use the dimensions given in Table 2. Flanges to be ring type.

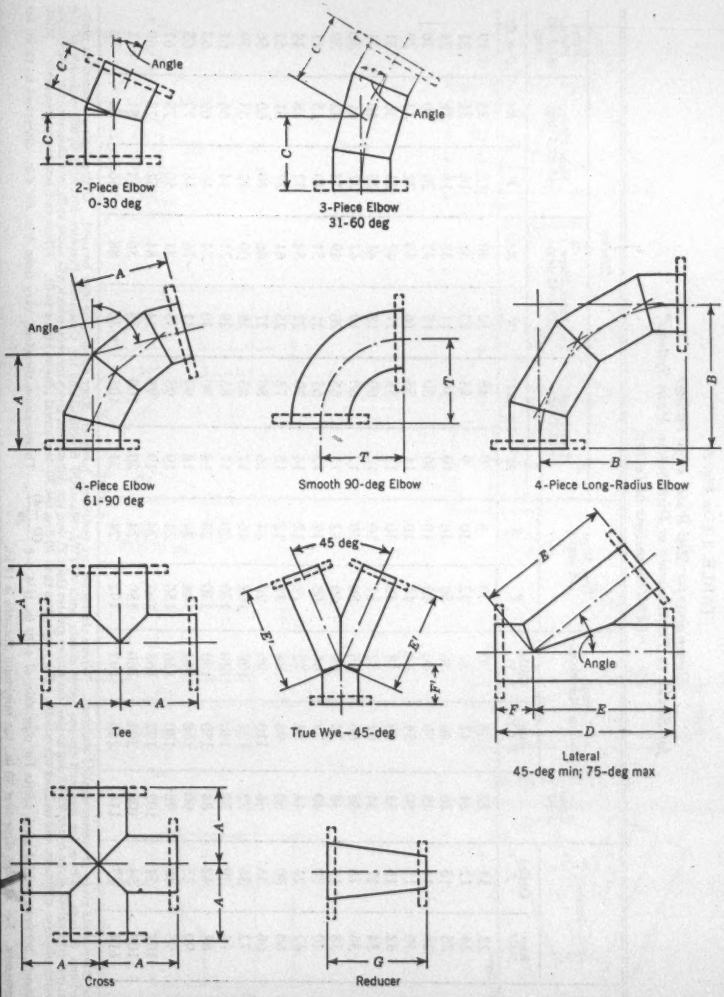


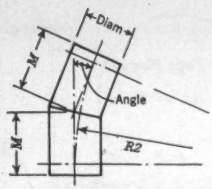
Fig. 1. Steel Water Pipe Fittings, Class 125, for Service in Piping Systems in Water Works or Industrial Plants

TABLE 2 (*See Fig. 2*)

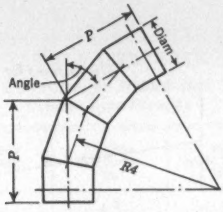
Dimensions for Steel Water Pipe Fittings
*for Service in Transmission or Distribution Main Systems**
(All Dimensions in Inches)

Nominal Diam.	Tee		Cross	Lateral (30-deg Minimum)		True Wye (45 deg)		Elbows						
								Two-Piece (0–22.5 deg)		Three-Piece (22.5–45 deg)		Four-Piece (45–67.5 deg)		Five-Piece (67.5–90 deg)
	Run $J+J$	Outlet J	$J+J$	Run $X+Y$	Outlet Y	Y	X	M	$R2$	N	$R3$	P	$R4$	Q or $R5$
4	22	11	22	38	30	30	8	9	45	12	30	15	23	19
6	24	12	24	42	32	32	10	9	45	13	32	16	24	21
8	26	13	26	46	36	36	10	10	50	14	35	18	27	24
10	28	14	28	50	40	40	10	10	50	15	37	20	30	26
12	30	15	30	54	44	44	10	11	55	16	40	22	33	29
14	32	16	32	58	48	48	10	11	55	17	42	24	36	31
16	34	17	34	62	52	52	10	12	60	18	44	26	40	34
18	36	18	36	66	56	56	10	12	60	19	47	27	41	36
20	38	19	38	72	60	60	12	13	65	20	49	28	42	39
22	40	20	40	78	66	66	12	13	65	21	51	30	45	41
24	42	21	42	84	72	72	12	14	70	22	54	32	48	44
30	60	30	60	96	84	84	12	15	75	25	61	37	51	51
36	66	33	66	110	96	96	14	16	80	27	66	40	60	56
42	72	36	72	124	108	108	16	17	85	28	69	42	63	59
48	78	39	78	140	120	120	20	17	85	29	71	44	66	61
54	84	42	84	150	130	130	20	18	90	30	74	46	69	63
60	90	45	90	168	144	144	24	18	90	31	76	48	72	65
66	96	48	96	180	156	156	24	19	95	33	81	50	75	67
72	102	51	102	192	168	168	24	19	95	34	84	52	78	69
78	108	54	108	204	180	180	24	20	100	35	86	54	81	71
84	114	57	114	216	192	192	24	21	105	36	88	56	84	73

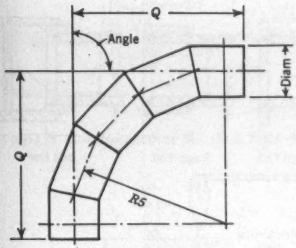
* The dimensions given are recommended standard. Where clearances are restricted, however, other dimensions may be used. Dimensions are based on plain- or flanged-end fittings. Those for tees and crosses agree closely with AWWA Standard Specifications for Cast-Iron Pressure Fittings, C100, exclusive of the bell. Lateral dimensions given cover 30–75 deg angles; for angles greater than 75 deg, use dimension, given for tees, which will allow sufficient clearance for fabrication of reinforcing plates or stiffeners, and welding operations. For angles less than 30 deg, use 30-deg lateral plus an elbow to fit. Elbow radiuses are approximate. Minimum distance between welded girth joints for pipe sizes of 4–30 in. is $4 + 8\frac{(D-4)}{26}$. These values, therefore, vary from 4 in. for 4-in diameters to 12 in. for 30-in. diameters. For all pipe sizes over 30 in., the minimum distance is 12 in.



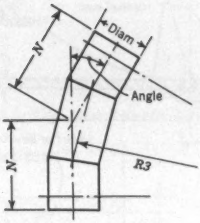
2-Piece Elbow
0-22.5 deg



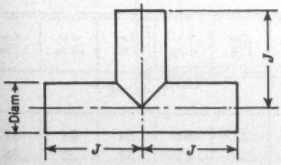
4-Piece Elbow
45-67.5 deg



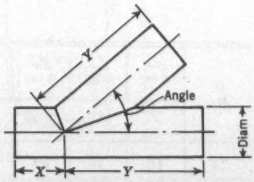
5-Piece Elbow
67.5-90 deg



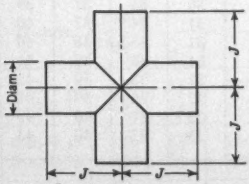
3-Piece Elbow
22.5-45 deg



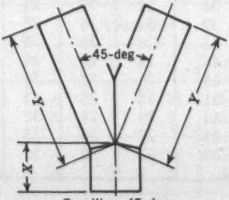
Tee



Lateral
30-deg min; 75-deg max



Cross

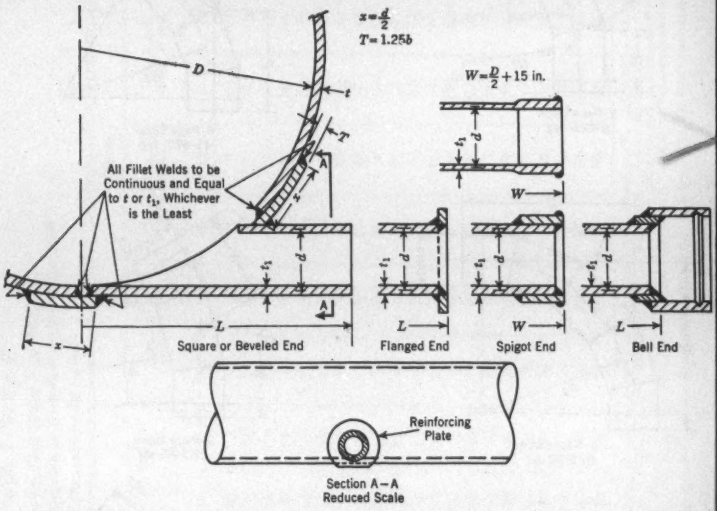


True Wye – 45-deg

Fig. 2. Steel Water Pipe Fittings for Service in Transmission or Distribution Main Systems

TABLE 3

Dimensions for Welded-Steel Tangent-Type Blowoffs



D in.	d = 4 in.	d = 6 in.	d = 8 in.	d = 10 in.	d = 12 in.	D in.	d = 4 in.	d = 6 in.	d = 8 in.	d = 10 in.	d = 12 in.
	L—in.						L—in.				
24	19	21	22	22	22	63	28	31	34	35	37
27	20	22	24	24	24	66	28	31	34	36	38
30	21	23	25	25	25	69	29	32	35	37	39
33	22	24	26	26	26	72	29	33	35	38	39
36	22	25	27	28	28	75	30	33	36	39	40
39	23	26	28	29	29	78	30	34	37	39	41
42	24	26	28	30	30	81	31	34	37	40	41
45	24	27	29	31	31	84	31	35	38	40	42
48	25	28	30	32	33	87	31	35	38	41	43
51	25	28	31	32	33	90	32	36	39	42	43
54	26	29	32	33	34	93	32	36	39	42	44
57	27	30	33	34	35	96	33	37	40	43	45
60	27	30	33	35	36	99	33	37	40	43	45

D is the inside diameter of the pipeline, and d *is the nominal diameter of the blowoff connection.* L *may be varied slightly to suit specific requirements. For details of flanges, see AWWA C207. For details of bells and spigots, see Table 4.*

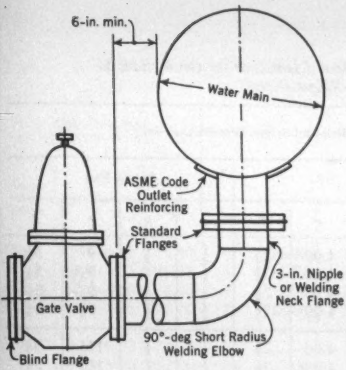


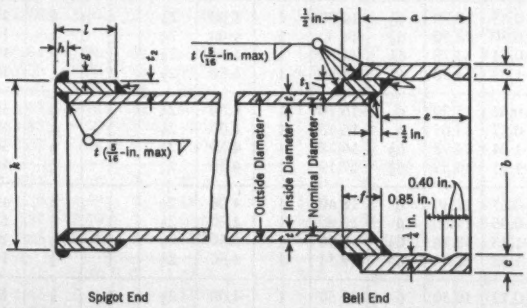
TABLE 3 ALTERNATE

Dimensions for Welded-Steel Vertical-Type Blowoffs

Nominal Pipe Size* in.	Outside Diameter in.	Pipe Thickness in.
4	4.5	0.337
6	6.63	0.432
8	8.63	0.322
12	12.75	0.375

* The 4-in. and 6-in. pipe is extra strength; the 8-in. and 12-in. is standard.

TABLE 4—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe**



Cast-Iron Pipe Dimensions—in.			Steel Bell-and-Spigot Dimensions—in.									
			Bell End					Spigot End				
Nominal Diam.	Thickness	Outside Diam.	*a*	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
3	0.37–0.45	3.80	5½	4.60	¾	3.50	2½	5/16	⅜	3¼	5
	0.46–0.53	3.96	5½	4.76	¾	3.50	2½	5/16	⅜	4	5
4	0.40–0.45	4.80	5½	5.60	¾	3.50	2½	5/16	⅜	4¼	5
	0.46–0.55	5.00	5½	5.80	¾	3.50	2½	5/16	⅜	5	5
6	0.43–0.50	6.90	5½	7.70	¾	3.50	2½	5/16	⅜	6¾	5
	0.51–0.60	7.10	5½	7.90	¾	3.50	2½	5/16	⅜	7⅛	5
	0.61–0.66	7.22	6	8.02	¾	4.00	2½	5/16	⅜	7¼	5½
	0.67–0.74	7.38	6	8.18	¾	4.00	2½	5/16	⅜	7⅜	5½

* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 4 (*contd.*)—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe* *

Cast-Iron Pipe Dimensions—*in.*			Steel Bell-and-Spigot Dimensions—*in.*								
Nominal Diam.	Thickness	Outside Diam.	Bell End					Spigot End			
			a	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
8	0.46–0.57	9.05	6	9.85	1	4.00	2½	¼	½	9	5½
	0.58–0.70	9.30	6	10.10	1	4.00	2½	¼	½	9 5/16	5½
	0.71–0.76	9.42	6	10.22	1	4.00	2½	¼	½	9 7/16	5½
	0.77–0.85	9.60	6	10.40	1	4.00	2½	¼	½	9⅝	5½
10	0.50–0.60	11.10	6	11.90	1	4.00	2½	¼	½	11⅛	5½
	0.61–0.75	11.40	6	12.20	1	4.00	2½	¼	½	11⅜	5½
	0.76–0.85	11.60	6½	12.40	1	4.50	2½	¼	½	11⅝	6
	0.86–0.97	11.84	6½	12.64	1	4.50	2½	¼	½	11⅞	6
12	0.54–0.65	13.20	6½	14.00	1	4.00	2½	¼	½	13½	6
	0.66–0.80	13.50	6½	14.30	1	4.00	2½	¼	½	13½	6
	0.81–0.94	13.78	6½	14.58	1	4.50	2½	¼	½	13¾	6
	0.95–1.09	14.08	6½	14.88	1	4.50	2½	¼	½	14⅛	6
14	0.54–0.62	15.30	6	16.10	1	4.00	2½	¼	½	15 5/16	5½
	0.63–0.87	15.65	6	16.45	1	4.00	2½	¼	½	15⅝	5½
	0.88–1.04	15.98	6½	16.78	1	4.50	2½	¼	½	16	6
	1.05–1.21	16.32	6½	17.12	1	4.50	2½	¼	½	16 5/16	6
16	0.58–0.67	17.40	6	18.40	1	4.00	2½	¼	½	17⅜	5½
	0.68–0.95	17.80	6	18.80	1	4.00	2½	¼	½	17¾	5½
	0.96–1.13	18.16	6½	18.96	1	4.50	2½	¼	½	18 5/16	6
	1.14–1.32	18.54	6½	19.34	1	4.50	2½	¼	½	18½	6
18	0.63–0.72	19.50	6	20.50	1	4.00	2½	¼	½	19½	5½
	0.73–1.01	19.92	6	20.92	1	4.00	2½	¼	½	19 15/16	5½
	1.02–1.22	20.34	6½	21.14	1	4.50	2½	¼	½	20 5/16	6
	1.23–1.44	20.78	6½	21.58	1	4.50	2½	¼	½	20¾	6
20	0.66–0.82	21.60	6	22.60	1	4.00	2½	¼	½	21⅝	5½
	0.83–1.08	22.06	6	23.06	1	4.00	2½	¼	½	22 5/16	5½
	1.09–1.32	22.54	6½	23.34	1	4.50	2½	¼	½	22½	6
	1.33–1.56	23.02	6½	23.82	1	4.50	2½	¼	½	23	6

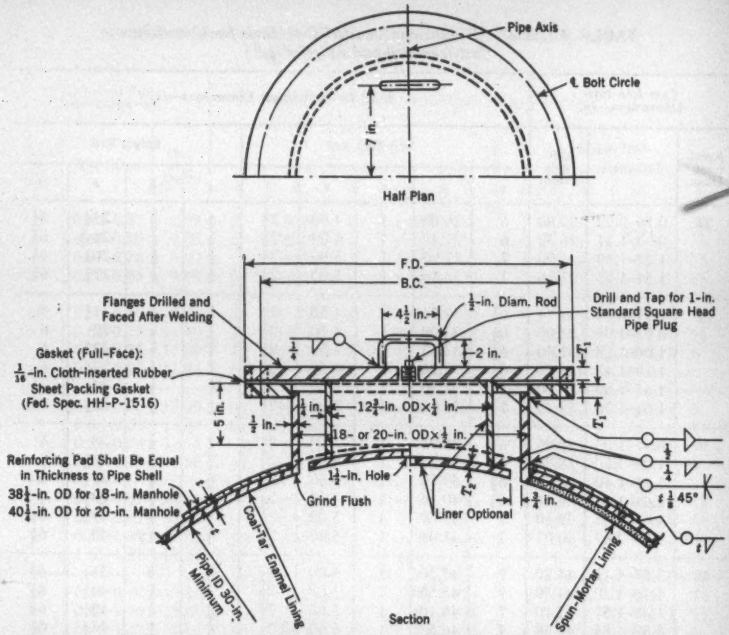
* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 4 (*contd.*)—*Dimensions for Steel Pipe Ends for Connections to Cast-Iron Bell-and-Spigot Pipe**

Cast-Iron Pipe Dimensions—*in.*			Steel Bell-and-Spigot Dimensions—*in.*								
			Bell End					Spigot End			
Nominal Diam.	Thickness	Outside Daim.	*a*	*b*	*c*	*e*	*f*	*g*	*h*	*k*	*l*
24	0.74–0.92	25.80	6	26.80	1	4.00	2½	¼	½	25¾	5½
	0.93–1.21	26.32	6	27.32	1	4.00	2½	¼	½	26 5/16	5½
	1.22–1.50	26.90	7	27.90	1	5.00	2½	¼	½	26⅞	6½
	1.51–1.93	27.76	7	28.56	1	5.00	2½	¼	½	27¾	6½
30	0.87–0.92	31.74	6½	32.74	1	4.50	2½	¼	½	31¾	6
	0.93–1.05	32.00	6½	33.00	1	4.50	2½	¼	½	32	6
	1.06–1.18	32.40	6½	33.40	1	4.50	2½	¼	½	32⅜	6
	1.19–1.42	32.74	6½	33.74	1	4.50	2½	¼	½	32¾	6
	1.43–1.60	33.10	7	34.10	1	5.00	2½	¼	½	33½	6½
	1.61–1.78	33.46	7	34.46	1	5.00	2½	¼	½	33½	6½
36	0.97–1.03	37.96	6½	38.96	1	4.50	2½	¼	½	38	6
	1.04–1.20	38.30	6½	39.30	1	4.50	2½	¼	½	38¼	6
	1.21–1.40	38.70	6½	39.70	1	4.50	2½	¼	½	38¾	6
	1.41–1.63	39.16	6½	40.16	1	4.50	2½	¼	½	39½	6
	1.64–1.85	39.60	7	40.60	1	5.00	2½	¼	½	39⅝	6½
	1.86–2.07	40.04	7	41.04	1	5.00	2½	¼	½	40	6½
42	1.07–1.15	44.20	7	45.20	1	5.00	2½	¼	½	44¼	6½
	1.16–1.30	44.50	7	45.50	1	5.00	2½	¼	½	44½	6½
	1.31–1.57	45.10	7	46.10	1	5.00	2½	¼	½	45⅛	6½
	1.58–1.84	45.58	7	46.58	1	5.00	2½	¼	½	45⅝	6½
48	1.18–1.30	50.50	7	51.50	1	5.00	2½	¼	½	50½	6½
	1.31–1.45	50.80	7	51.80	1	5.00	2½	¼	½	50¾	6½
	1.46–1.72	51.40	7	52.40	1	5.00	2½	¼	½	51⅜	6½
	1.73–2.04	51.98	7	52.98	1	5.00	2½	¼	½	52	6½
54	1.30–1.38	56.66	7½	57.66	1	5.50	2½	¼	½	56⅜	7
	1.39–1.60	57.10	7½	58.10	1	5.50	2½	¼	½	57½	7
	1.61–1.89	57.80	7½	58.80	1	5.50	2½	¼	½	57¾	7
	1.90–2.25	58.40	7½	59.40	1	5.50	2½	¼	½	58⅜	7
60	1.39–1.45	62.80	7½	63.80	1	5.50	2½	¼	½	62¾	7
	1.46–1.75	63.40	7½	64.40	1	5.50	2½	¼	½	63⅜	7
	1.76–2.15	64.20	7½	65.20	1	5.50	2½	¼	½	64½	7
	2.16–2.46	64.82	7½	65.82	1	5.50	2½	¼	½	64⅞	7

* Dimensions are based on American Standard Specifications for Cast-Iron Pit-Cast Pipe for Water or Other Liquids—ASA A21.2 (AWWA C102) and AWWA C100.

TABLE 5—*Dimensions for 18-in. and 20-in. Welded-Steel Manholes* *



Working Pressure† psi	F.D. in.	B.C. in.	T_1 in.	Number of Bolts‡	Bolt Size—in.	Bolt Hole Diam.—in.
18-in. Manhole						
150	25	22 3/4	1 1/16	16	1 1/8	1 1/4
175	25	22 3/4	1 1/16	16	1 1/8	1 1/4
225	28	24 3/4	1 1/8	24	1 1/4	1 3/8
250	28	24 3/4	1 1/8	24	1 1/4	1 3/8
300	28	24 3/4	1 1/8	24	1 1/4	1 3/8
20-in. Manhole						
150	27 1/2	25	1 1/8	20	1 1/8	1 1/4
175	27 1/2	25	1 1/8	20	1 1/8	1 1/4
225	30 1/2	27	1 1/8	24	1 1/4	1 3/8
250	30 1/2	27	1 1/16	24	1 1/4	1 3/8
300	30 1/2	27	1 1/8	24	1 1/4	1 3/8

* T_3 is 1/2 in. or may be coupon cut from main cylinder. For pipe slopes greater than 5 deg, the riser pipe is to be set vertical. The inside face of the manhole is to be flush with the finished interior pipe surface. F.D. stands for flange dimension, B.C. for bolt circle.

† Working pressure is to be stamped on manhole by the fabricator with 1/4-in. dies.

‡ Hexagonal bolts and nuts are to be used.

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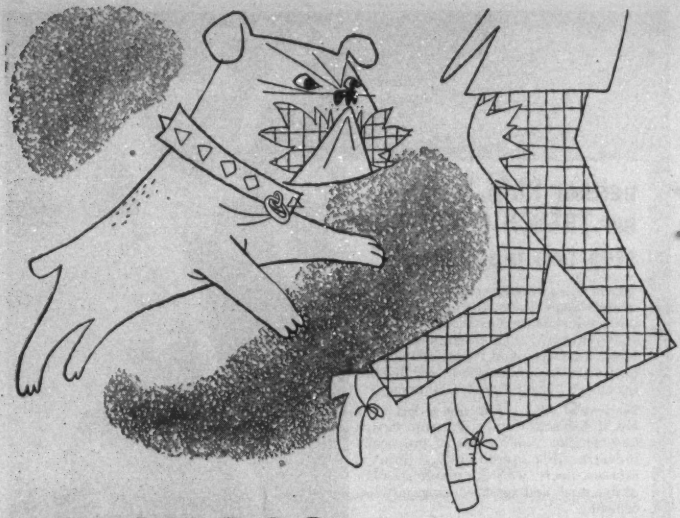

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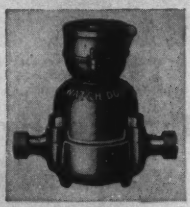

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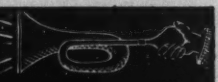

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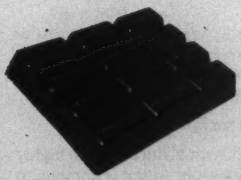

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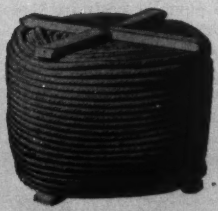

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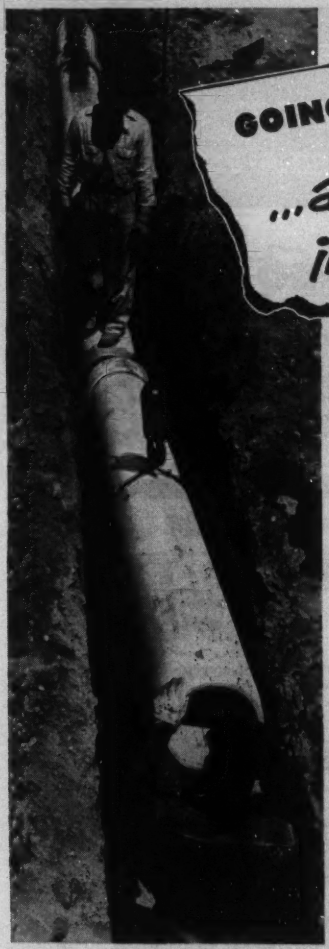

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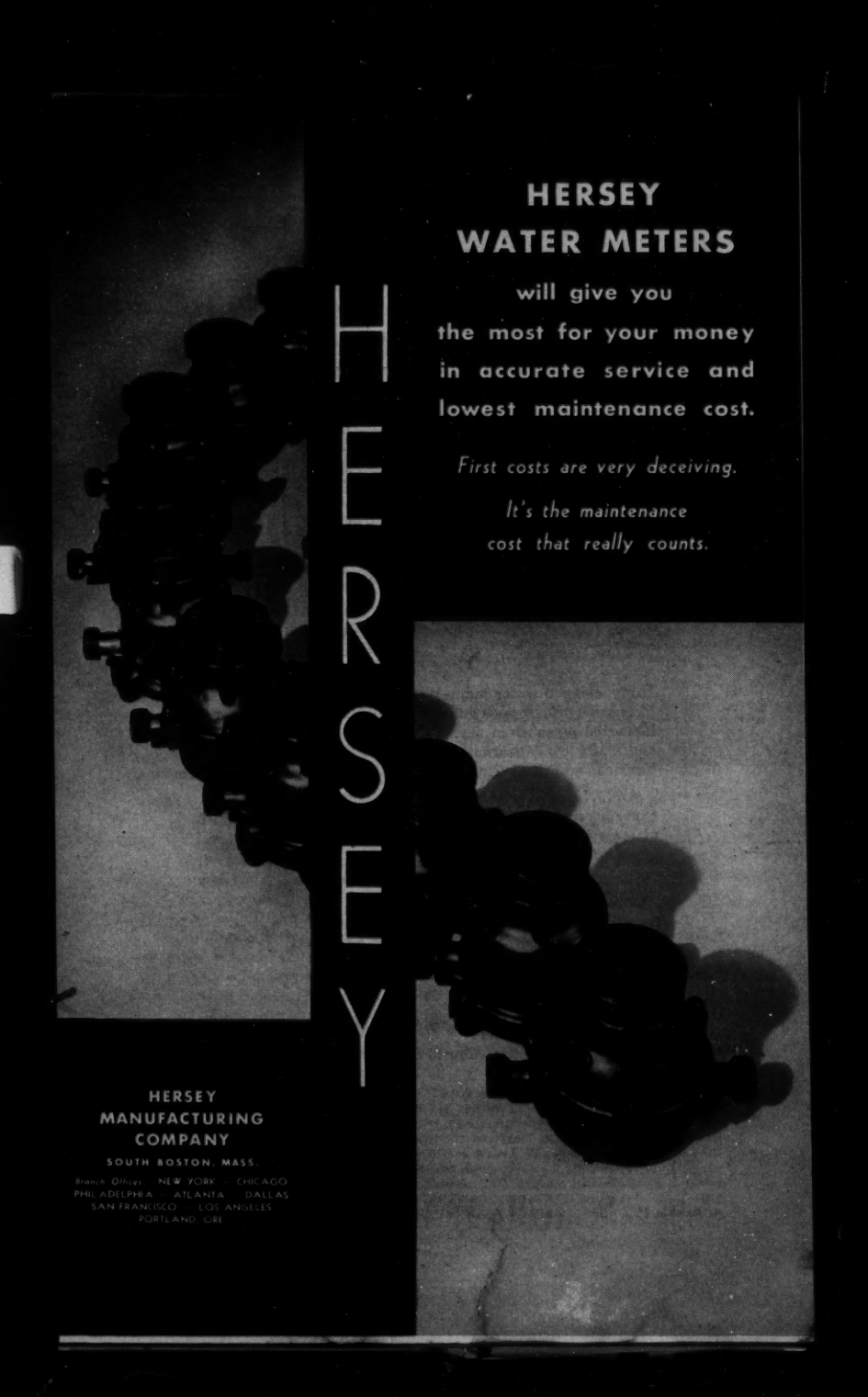
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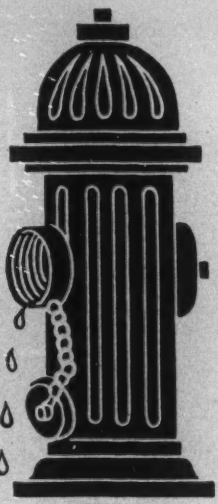

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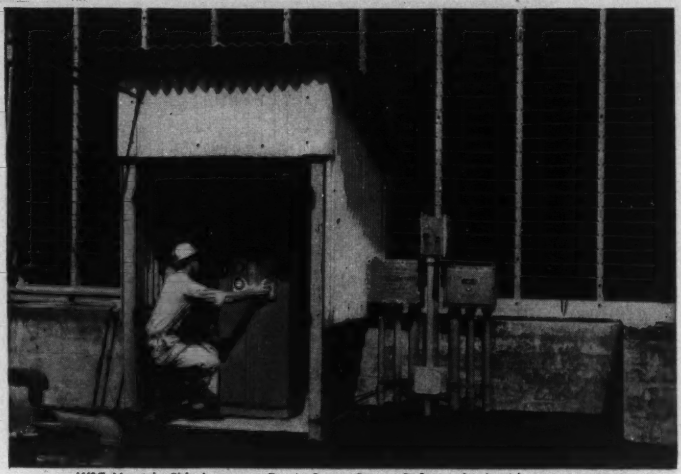

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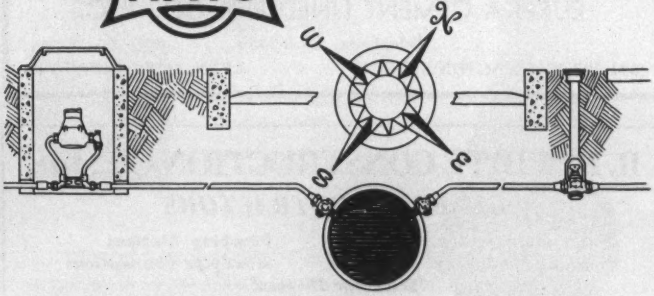

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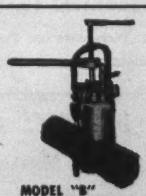
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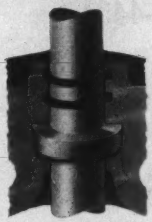

O-RING SEALS

Provide excellent seal, prevent binding of stem and result in very easy valve operation. KENNEDY O-Rings are located above collar, permit repacking under pressure. Optional on KENNEDY non-rising stem A.W.W.A. valves.

STRONGER CONSTRUCTION

Rugged design and construction features iron that is 50% stronger than ordinary cast iron to keep KENNEDY Valves doing a better job over a longer, more dependable operating life.

KENNEDY Fig. 56
A.W.W.A. Standard Iron-Body Double-Disc Gate Valve, with Bell Ends.

KENNEDY manufactures a complete line of water works valves. Sizes range from 2" to 48" including non-rising stem and outside-screw-and-yoke valves. These valves feature the hook-and-wedge type disc mechanism for easier, better and more positive valve operation. A wide variety of pipe connections are available.

For maximum service life, for the greatest value for your valve dollar, specify KENNEDY valves and fire hydrants. Remember, KENNEDY means dependability in valves, fire hydrants and accessories.

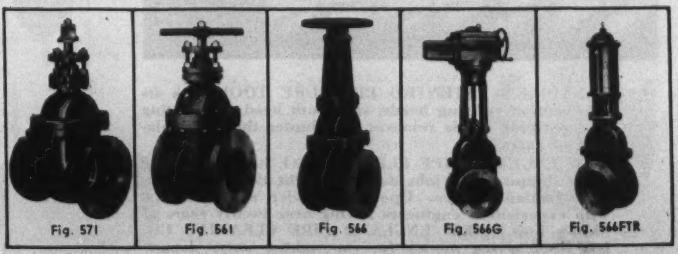

Fig. 571 | Fig. 561 | Fig. 566 | Fig. 566G | Fig. 566FTR

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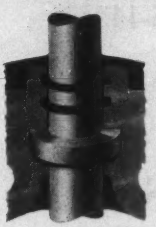
Give You More Value For Your Valve Dollar . . .



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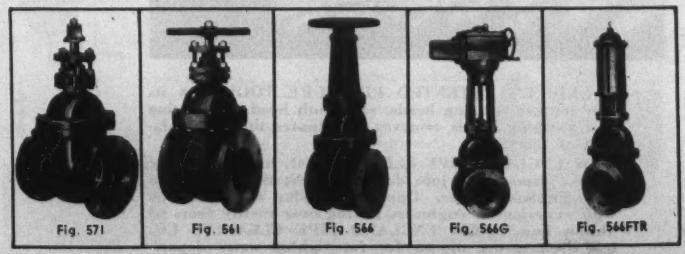

Fig. 571 | Fig. 561 | Fig. 566 | Fig. 566G | Fig. 566FTR

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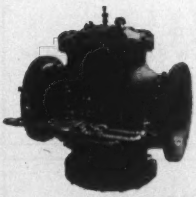
AUTOMATIC VALVES



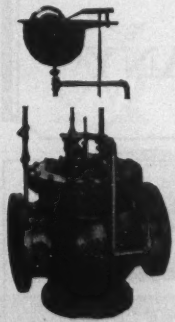
Controls elevation of water in tanks, basins and reservoirs

ALTITUDE VALVE
1. Single Acting
2. Double Acting

Maintains desired discharge regardless of change in rate of flow pressure

REDUCING VALVE

Regulates pressures in gravity and pump systems; between reservoirs and zones of different pressures, etc.



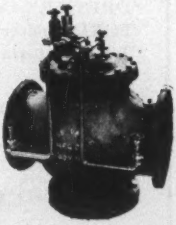
Maintains levels in tank, reservoir or basin

1. As direct acting.

2. Pilot operated and with float traveling between two stops, for upper and lower limit of water elevation.

FLOAT VALVE

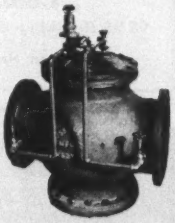
A self contained unit, with controls three or more automatic



COMBINATION VALVE

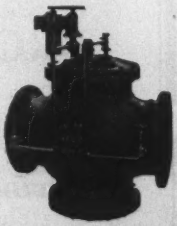
Combination automatic control both directions through the valve.

Maintains safe operating pressures for conduits, distribution and pump discharge



SURGE-RELIEF VALVE

Electric remote control—solenoid or motor can be furnished



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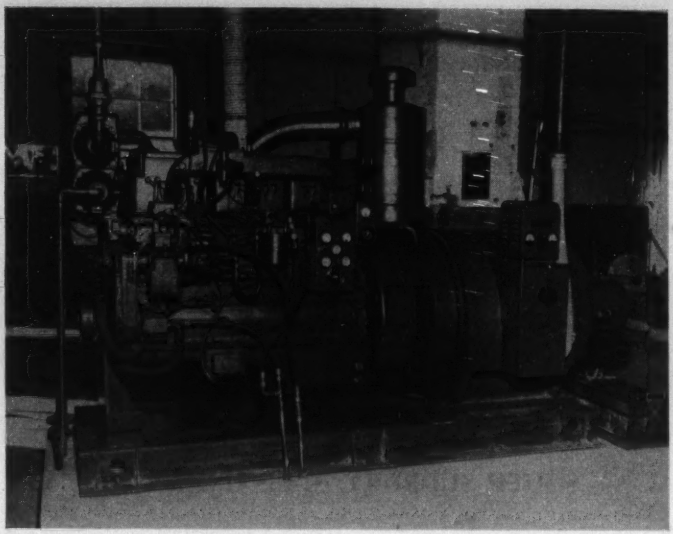

CUMMINS MODEL NHRSGA-150 KW GENERATOR SET

Rings Corner Water Pumping Station
Town of Amesbury, Massachusetts

Improvements at the Rings Corner Water Pumping Station were engineered by Camp, Dresser and McKee of Boston, and included the installation of a 150 KW Cummins generator set because of the long power outages experienced by other towns during recent hurricanes.

The generator set was selected to handle a total load of 122 HP, consisting of one 75 HP and one 30 HP motor, plus small motors and lighting. It is capable of starting the 75 HP motor with all other connected load on the line. Tests of the Cummins 150 KW generator set (rated at 40° ambient and 20% inherent voltage regulation) have shown excellent results and ample capacity for the above operation.

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Ball-and-socket joint cast iron pipe for water main crossing river.

Where installations are planned for long-term service to assure low cost per service year, water works engineers rely on cast iron pipe as a dependable and adaptable material. Consequently, it is specified for a wide variety of applications, both utility and industrial, including water supply, sewerage, fire protection, process industries and many forms of special construction. Long life and low maintenance cost are *proved* results of the high beam-strength, compressive-strength, shock-strength and effective resistance to corrosion of cast iron pipe. Cast Iron Pipe Research Association, Thos. F. Wolfe, Managing Director, 122 So. Michigan Ave., Chicago 3, Ill.

Cast iron water main still functioning in Boston after a century of service.

CAST IRON PIPE SERVES FOR CENTURIES

a perfect fit!

The ingenious "Tyton Joint" is simple, speedy and sure. A specially designed rubber gasket fits into the bell end of the receiving pipe. The connecting pipe slides easily into place, compressing the gasket, which provides a tight and lasting seal.

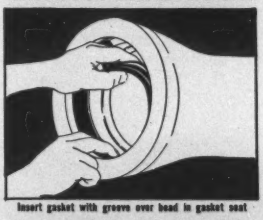
"Tyton Joint" is remarkably easy to install. No bell holes. Can be laid in rain or wet trench. Even an inexperienced crew masters the know-how quickly.

"AH WARNED ONCLE RAFE NOT TO STICK HIS HEAD IN, GRAN'MAW..."

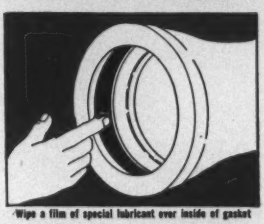
U.S. PIPE FOR WATER, SEWERAGE AND

TYTON

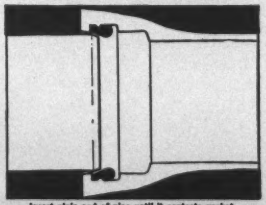
ONLY FOUR SIMPLE ACTIONS



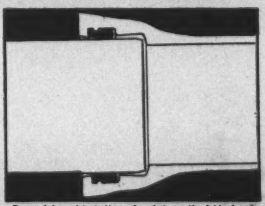
Insert gasket with groove over bead in gasket seat



Wipe a film of special lubricant over inside of gasket



Insert plain end of pipe until it contacts gasket



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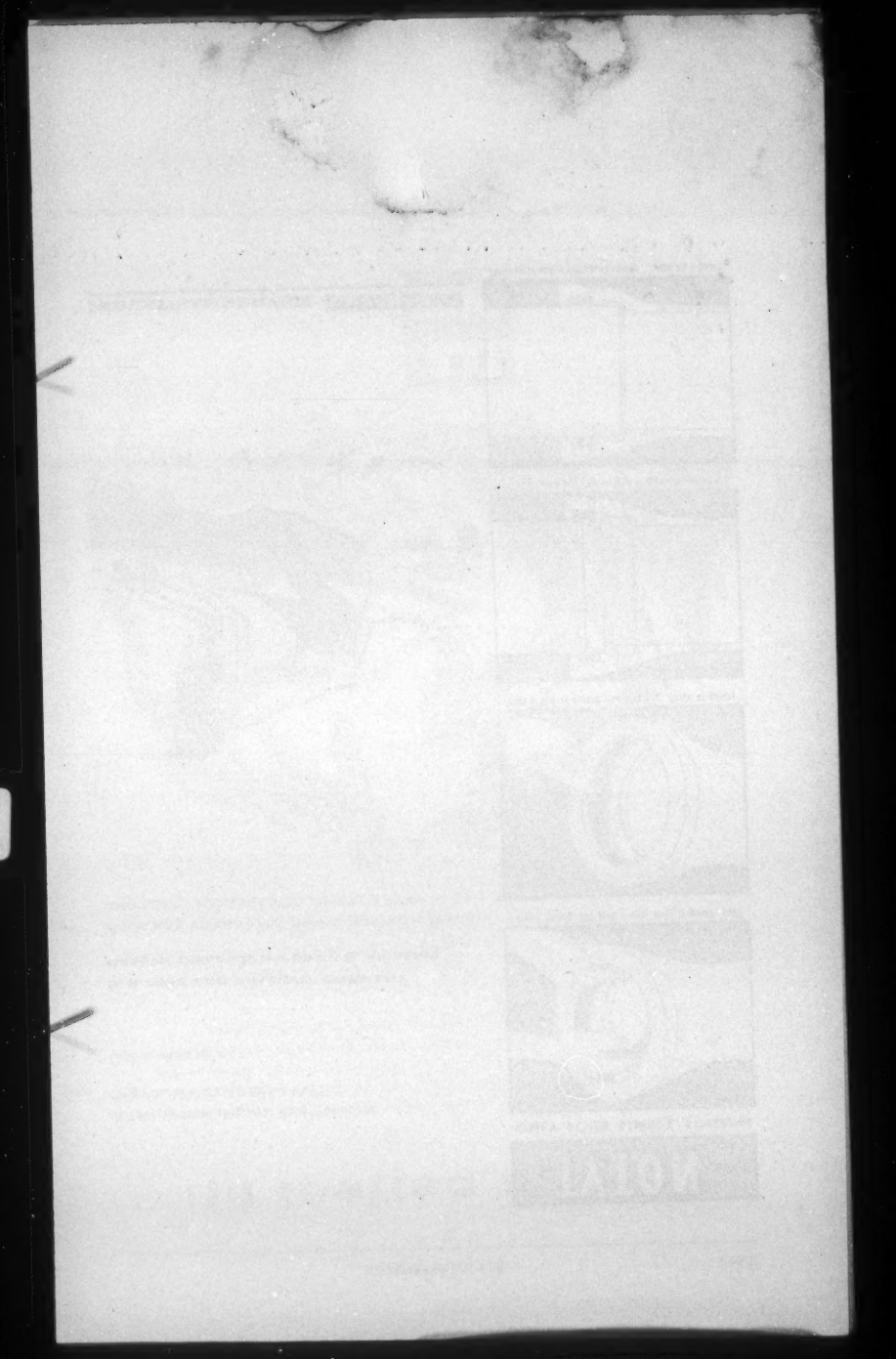

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